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An Investigation of

Reinforced Steel Columns

Theoretical and Applied Mechanics

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AN INVESTIGATION OF REINFORCED STEEL COLUMNS

BY

ARTHUR RUSSELL LORD

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I HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

ARTHUR RUSSELL LORD

ENTITLED AN INVESTIGATION OF REINFORCED STEEL COLUMNS

BE ACCEPTED AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Master of Science in Theoretical and Applied Mechanics

A. N. Talbot
In Charge of Major Work

A. N. Talbot
Head of Department

Recommendation concurred in:

Committee

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REINFORCED STEEL COLUMN

EXTENSOMETERS IN POSITION

AN INVESTIGATION OF

REINFORCED STEEL COLUMNS

TABLE OF CONTENTS

I INTRODUCTION

III EXPERIMENTAL DATA AND DISCUSSION

A PLAIN STEEL COLUMNS - EFFECT OF LENGTH

B REINFORCED STEEL COLUMNS - EFFECT OF LENGTH

C REINFORCED STEEL COLUMNS - VALUE OF REINFORCEMENT

D REINFORCED STEEL COLUMNS – FIREPROOFING

E REINFORCED STEEL COLUMNS WITH SPIRALS

F COMPARISON OF RESULTS WITH OTHER TESTS

DIAGRAMS

TABLES

I

INTRODUCTION

AN INVESTIGATION OF
REINFORCED STEEL COLUMNS

I INTRODUCTION

1 - <u>Use of Reinforced Steel Columns.</u> Reinforced concrete building construction has reached a stage in its development where its advantages and disadvantages are becoming well known. It is now tending to assume types more or less fixed and to attain fixed standards of design. The limitations of current design are also coming to be recognized and methods are being sought by which such limitations may be overcome. In the case of very high or very heavily loaded buildings the columns tend to assume prohibitive size. In office buildings they can no longer be concealed in partitions, and in warehouses they occupy much space and obstruct free passage. At present this difficulty is overcome in part by the use of extremely high unit working stresses, warrant for this use being based on the presence of a large amount of spiral reinforcement. Tests show that

such reinforcement does furnish the required factor of safety of column strength, but they show no less conclusively that the excessive deformation which occurs in columns so reinforced before this carrying capacity is realized, makes the use of these high working stresses questionable on account of the effect on other parts of the structure. When this is more fully realized, it would seem that for certain types of buildings some substitute must be found for our present reinforced concrete column, which will have grown to excessive dimensions.

Previous to the introduction of reinforced concrete the standard building column was of structural steel, and this type remains today for many purposes the favored and best adapted construction. In size it is entirely satisfactory. Its cost is somewhat higher than that of the reinforced concrete column. It must also be protected from damage by rust and by fire. During the past few years a tendency has developed among designers to replace the large reinforced concrete column by the relatively small structural steel column. In doing this the fire protection of tile commonly associated with steel construction has been replaced by a solid core and covering of concrete. The properties of a column so constituted, in which the steel forms a considerable portion of the volume and affords the major portion of the load carrying capacity, have been but slightly investigated and at the present time but little data exists on which to base intelligent design. The object of the tests herein

described was to provide such data on one particular section and type of column. The attention has been concentrated on one section and type in the belief that a thoro testing of one type is of more value that an equal effort divided among several types of column.

2 – Present Standards of Design. In the design of a structural steel column, filled with concrete, there exist two points of view quite sharply differentiated in principle. From one point of view the concrete simply affords to the steel protection from fire and corrosion. It is maintained that the additional strength afforded by the concrete is not considerable in amount and is not available for design. Holding this view it would seem most logical to employ a good quality of cinder concrete for column protection, because of its superior fire resisting qualities. From the second point of view it would seem that if the concrete be present it must act in unison with the steel and that its strengthening effect and its effect on the permissible deformation of the column should be taken into account. It is maintained that any load applied to the column will be divided between the steel and the concrete and hence the column must properly be designed as a reinforced steel column. Our present building codes afford little comfort to designers holding this second view point – almost universally it is ruled that that columns of structural steel filled with concrete, in which the steel forms eight per cent or over of the core section, shall be designed as steel columns without allowance

for the concrete. In a few cases no limitation of this nature is specified, but full steel stresses are permitted if the concrete is not considered, while only ten or twelve times a very low concrete stress is permitted in the steel if the concrete is considered as stressed. This latter provision is no less drastic than the explicit prohibition of dependence upon a strength in the concrete. Our present buildings in which reinforced steel columns have been used have generally been designed on the plain steel basis. The whole question involved may be stated in two parts: (1) Shall any allowance be made for the strengthening effect of the concrete in reinforced steel columns; and (2) If so, how great shall this allowance be and in what form shall it be expressed? The series of tests described in this thesis was planned to answer these questions for the particular type of column investigated. It is believed that the results are quite generally applicable, also, to other types not varying greatly from the one investigated. It is hoped that the fundamental principles underlying the design of reinforced steel columns may be better understood from a consideration of the data obtained by these tests and by such other tests as are available for comparison.

3 – Previously Published Test Data. The tests previously undertaken have been exceedingly limited and incomplete on account of the fact that they have been in all cases side excursions and not the main subject of the investigation. During the year 1908 ten columns composed of latticed angles were test-

ed at the University of Illinois. In this series there were many variables - per cent of reinforcement, arrangement of latticing, thickness of protective shell - so that no one feature was thoroly investigated and the results are not very conclusive. No plain steel columns were tested (without a concrete filling) and hence no good basis exists for computing the strengthening effect of the concrete. At the University of Wisconsin five steel columns of one type of latticed angle construction were tested in 1908, one being plain steel and four filled with concrete. The series is valuable but not sufficiently extended to satisfy. In both of the above mentioned series the percentage of steel was quite low, in no case exceeding six per cent, and the columns are more truly reinforced concrete than reinforced steel. In Germany Dr. von Emperger published in 1908 the results of a series of tests on specially designed steel columns. The investigation was intended to study simply the steel columns, but later it was decided to fill some of the tested columns (not appreciably bent) with concrete and retest them. The steel percentages in some cases were quite high and the tests are illuminating although very limited in number and somewhat lacking in completeness as published. The aforementioned tests will be discussed in succeeding paragraphs and the results compared with those obtained in this investigation. The brief statement given above discloses the scarcity of data which confronts the designer who wishes to design reinforced steel columns and secure their acceptance by the city building departments.

4 – Influences Affecting Test Results. In any discussion of tests and test data it is well to have in mind the effect which test conditions may have upon the results. In many cases such important information does not accompany the publication of the data. The speed at which the testing machine is operated during the test, the manner of applying the load, the condition of the ends of the specimen as affecting the distribution of load over the section, and the relative amount of fixidity at the two ends of the column are factors frequently very lightly regarded but which may have a very important bearing on the proper interpretation of the test data. In a succeeding paragraph the effect of these conditions will be more fully treated. At this point it is sufficient to note that these conditions must be held in mind in planning and in executing a series of experiments such as this, and also in applying the data found in the course of the investigation.

5 – Points Under Investigation in this Series of Tests. The investigation has been planned with the object in view of securing information on the following principal points, for the particular section and type of column tested: (a) the effect of length and slenderness on the strength of plain steel columns; (b) the effect of length upon the strength of similar columns filled with concrete; (c) the effect of filling the core with concrete of various mixtures and strengths; (d) the effect of adding a protective coat (fireproofing) of concrete upon the strength of the column, and the action of

this coat under load; (e) the effect of enclosing the column within a wire spiral upon the strength and stress-deformation relations of the column. As the total number of columns tested was only thirty-two it is evident that even the most careful planning could not provide for the complete answering of these questions, but it is believed that for this particular section and type of column a very satisfactory idea may be obtained of the effect of the conditions mentioned.

6 - Acknowledgments. The writer desires to express his appreciation of the action of the Illinois Steel Company in supplying the specimens required for a satisfactory investigation. The spirals for the six enclosed columns were furnished by the American System of Concrete Reinforcing of Chicago.

In the planning of the series of tests most helpful assistance has been rendered by Professor A. N. Talbot. The concreting of the specimens has been supervised by Mr. D. A. Abrams, and in the work of testing the writer was assisted a large part of the time by Mr. R. K. Steward. To all these gentlemen and to other members of the laboratory staff the writer desires to acknowledge his indebtedness for valuable assistance cheerfully rendered.

The tests were made in the 600 000 pound testing machine in the laboratory of applied mechanics of the University of Illinois, and constitute a portion of the work of the Engineering Experiment Station for the year 1910 - 1911.

Four of the six columns, reinforced with spirals, the strength of which exceeded the capacity of the University of Illinois testing machine, were sent to Lehigh University to be tested in their larger machine, in which a load of 830 000 pounds could be applied. This service was rendered freely, and the writer desires to acknowledge his especially indebtedness to the gentlemen of the Civil Engineering Department of Lehigh University, and to Mr. S. H. Ingberg in particular, for their valuable assistance in this matter.

II

MATERIALS, TEST PIECES,

AND METHODS OF TESTING

II MATERIALS, TEST PIECES, AND METHODS OF TESTING

7 - Materials and Their Properties. The materials used in the preparation of the test specimens were all of the grade employed in first class building construction. The properties in so far as they have been made the subject of special investigation are given in the following paragraphs.

Cement. The cement used was furnished by the Universal Portland Cement Company from stock and is representative of the reliable brands used in construction. Three samples were tested at various times during the progress of the construction of the specimens. The following table gives the results of the briquette tests, each result being the average of five briquettes tested.

Standard Briquette Tests of Cement.

Mixture	Tensile Strength in Pounds per Sq. Inch					
	Sample No. 1		Sample No. 2		Sample No. 3	
	7 da.	28 da.	7 da.	28 da.	7 da.	28 da.
Neat Cement	589	674	684	709	653	731
1 : 3 Standard Sand	198	278	227	283	240	319
1 : 3 Sand Used in Columns	265	323				

Fineness Test of Cement.

Sieve	Per Cent Passing
50	98.9
100	96.5
200	82.5

The initial set as determined by the Vicat Needle was observed to occur in 1 hour 20 minutes, and the final set in 4 hours 40 minutes. All the tests were passed in a manner to fully satisfy the standards adopted for cement in this country.

Sand. The sand used was torpedo sand from Attica, Indiana. It was of good quality, fairly sharp, clean, and well graded. It combined with the cement used in a very satisfactory manner giving a higher briquette test than did the same cement with standard sand (Ottawa testing sand). It was from the same locality and of the same quality as the sand used in making reinforced concrete test specimens for several years at the University of Illinois.

Stone. A good quality of rather hard limestone from Kankakee, Illinois, was used, being ordered to pass thru a 1-inch and over a 1/4-inch screen. It is representative of the stone most used in building construction of reinforced concrete in Illinois, and the same as has been used in the previous experimental work of the station. No special tests were made to determine its voids. In the columns tested the failure did not appear to result from the crushing or breaking of the stone in any case.

Proportions. Table 1 gives the proportions of the various materials used in the different batches of concrete from which thesis columns were made. The proportions are by weight in all cases - for the approximate proportions by parts or volumes see Table 3, page 28.

Table 1.

Concrete Proportions by Weight

Column	Proportions	Column	Proportions	Column	Proportions
8907	1:2.16:3.55	8923	1:2.04:3.54	8931	1:2.07:3.60
08	1:2.05:3.55	25	1:1.06:1.81	33	1:2.08:3.61
12	1:2.16:3.55	26	1:1.04:1.76	34	1:2.02:3.51
13	1:2.05:3.55	27	1:3.14:5.43	35	1:1.55:3.61
17	1:2.11:3.35	28	1:3.02:5.30	36	1:2.02:3.56
18	1:1.88:3.62	29	1:2.04:3.52	37	1:2.04:3.52
8922	1:2.09:3.42	8930	1:2.13:3.60	8938	1:2.08:3.67

Steel. The steel used in the manufacture of the columns was ordinary open-hearth structural steel. The columns were fabricated at the North Works of the Illinois Steel Company on the same basis as a commercial order. Two compression tests of flanges, composed of two 3 x 2 1/2 x 5/16 in. angles riveted back to back, thirty-two inches long, gave ultimate compressive strengths of 40 000 and 39 500 pounds per square inch. The stress-deformation curves began to bend at about 23 000 pounds per square inch and the modulus of elasticity as calculated from the straight portion of the stress-deformation curve was 27 400 000 and 32 400 000 pounds per square inch for the two specimens. Two similar flanges 7 ft. 10 in. long gave crushing strengths of 31 800 and 34 100 lb. per sq. in., straight line stress-deformation curves up to about 20 000 lb. per sq. in., and

a computed modulus of elasticity of 30 000 000 lb. per sq. in. for both specimens. The test observations are given in full on page 19, and the stress-deformation curves on pp. 20 to 23. The data is also tabulated in the general summary sheet (folded insert). A plain steel column of the full section two feet long gave a crushing strength of 37 500 lb. per sq. in. and a computed modulus of elasticity of 34 250 000 lb. per sq. in.

Tension tests of 1 1/2 x 5/16 x 18 in. specimens, cut from an untested flange, gave the following results.

Tension Tests of Steel.

Specimen No.	Ultimate Strength	Yield Point
1	57 800	37 800
2	67 600	41 800

(in lb. per sq. in.)

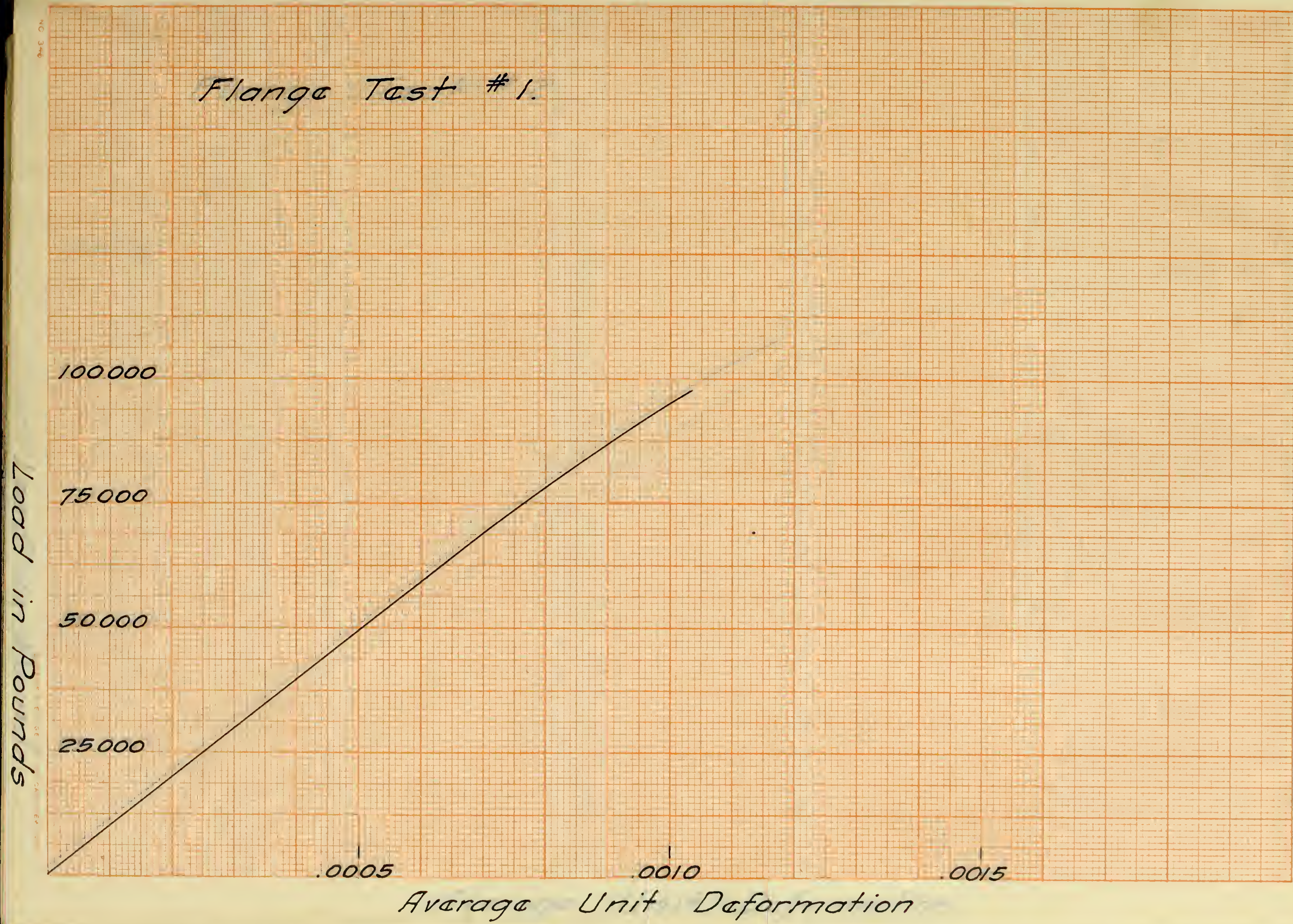
TESTS OF

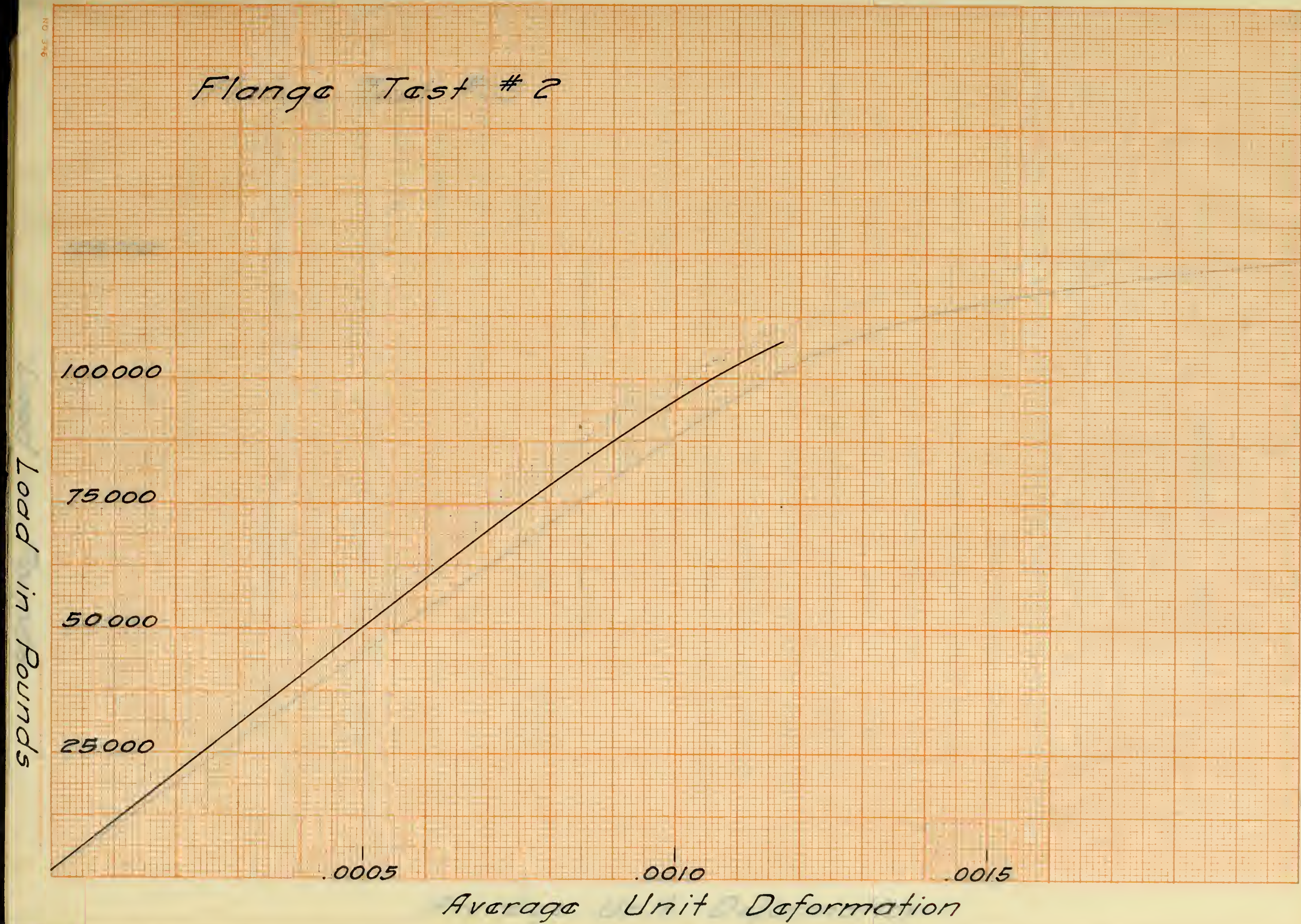
COMPRESSION SPECIMENS FROM FLANGES

No. 1		No. 2		No. 3		No. 4	
Load Applied	Average Unit Def.	Load	Unit Def.	Load	Unit Def.	Load	Unit Def.
1000	.00000	1000	.00000	600	.00000	500	.00000
5000	2	5000	4	5000	4	5500	1
10000	9	11000	8	15000	15	15200	12
15500	15	17000	16	25000	29	25000	21
20750	20	24700	24	35000	39	35600	32
30500	31	32500	31	45800	50	45500	42
38000	39	40500	40	55400	59	55300	52
45300	46	47500	47	65400	72	65200	61
53500	54	54700	55	75600	84	75300	71
60000	61	63000	63	85000	95	85600	84
67000	69	70500	71	95000	107	95000	93
74800	77	77200	78	105000	121	105200	108
82000	85	84800	86	115300	146	115000	126
90300	95	94000	99	125000	208	125000	169
97500	103	100000	106				
		107500	116				
103400	Ult.	110800	Ult.	128200	Ult.	130000	Ult.
Length:	94"	Length:	95"	Length:	32"	Length:	32 1/4"
L/r . . .	100	L/r . . .	100	L/r . . .	34.5	L/r . . .	34.7
G.L. . .	40"	G.L. . .	40"	G.L. . .	20"	G.L. . .	20"

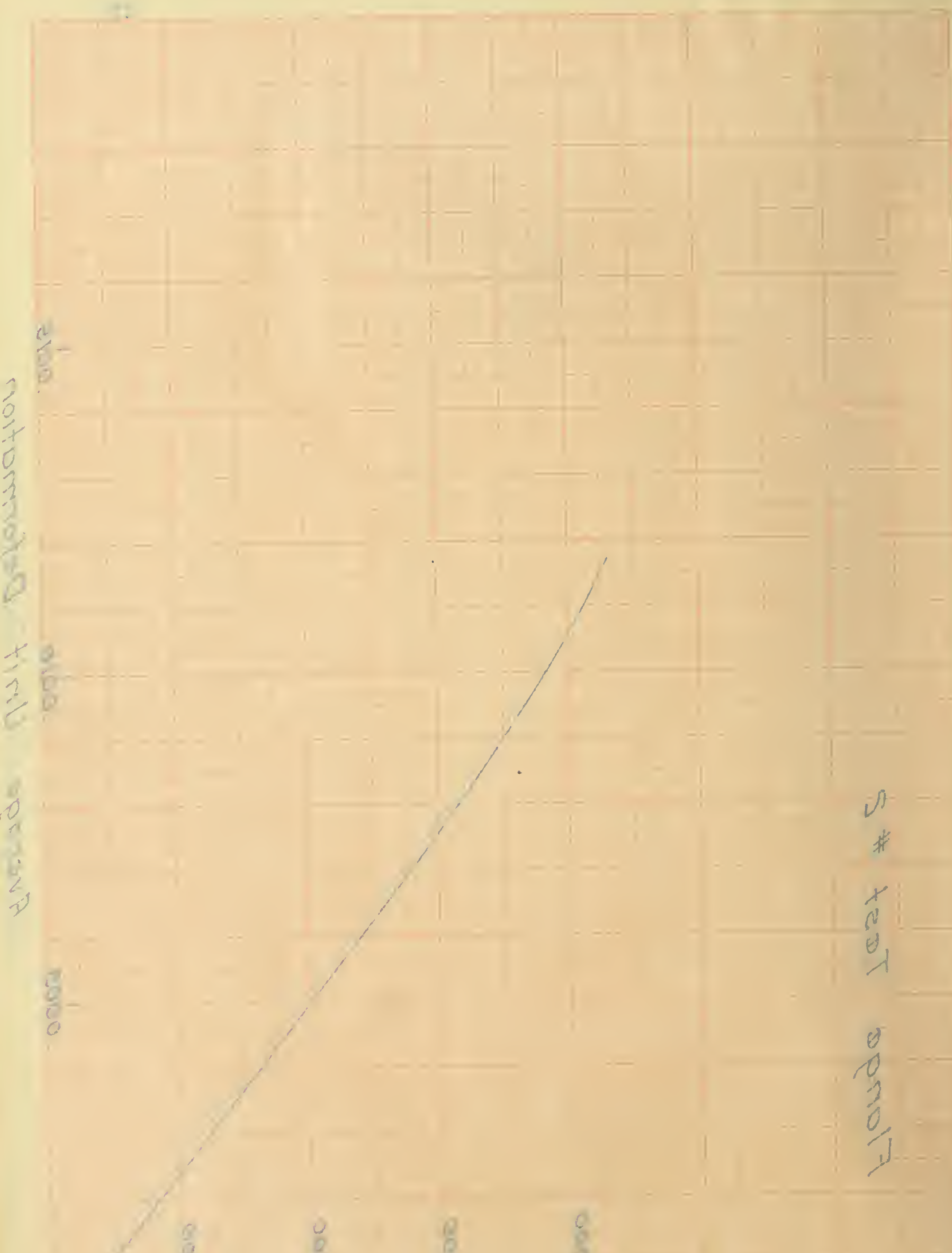
G.L. = Gauge Length

Failure of Nos. 1 and 2 occurred by bending and was very gradual. Bend was symmetrical about center of length. Failure of Nos. 3 and 4 occurred by straightening out of angles between tie plates and was quite gradual.

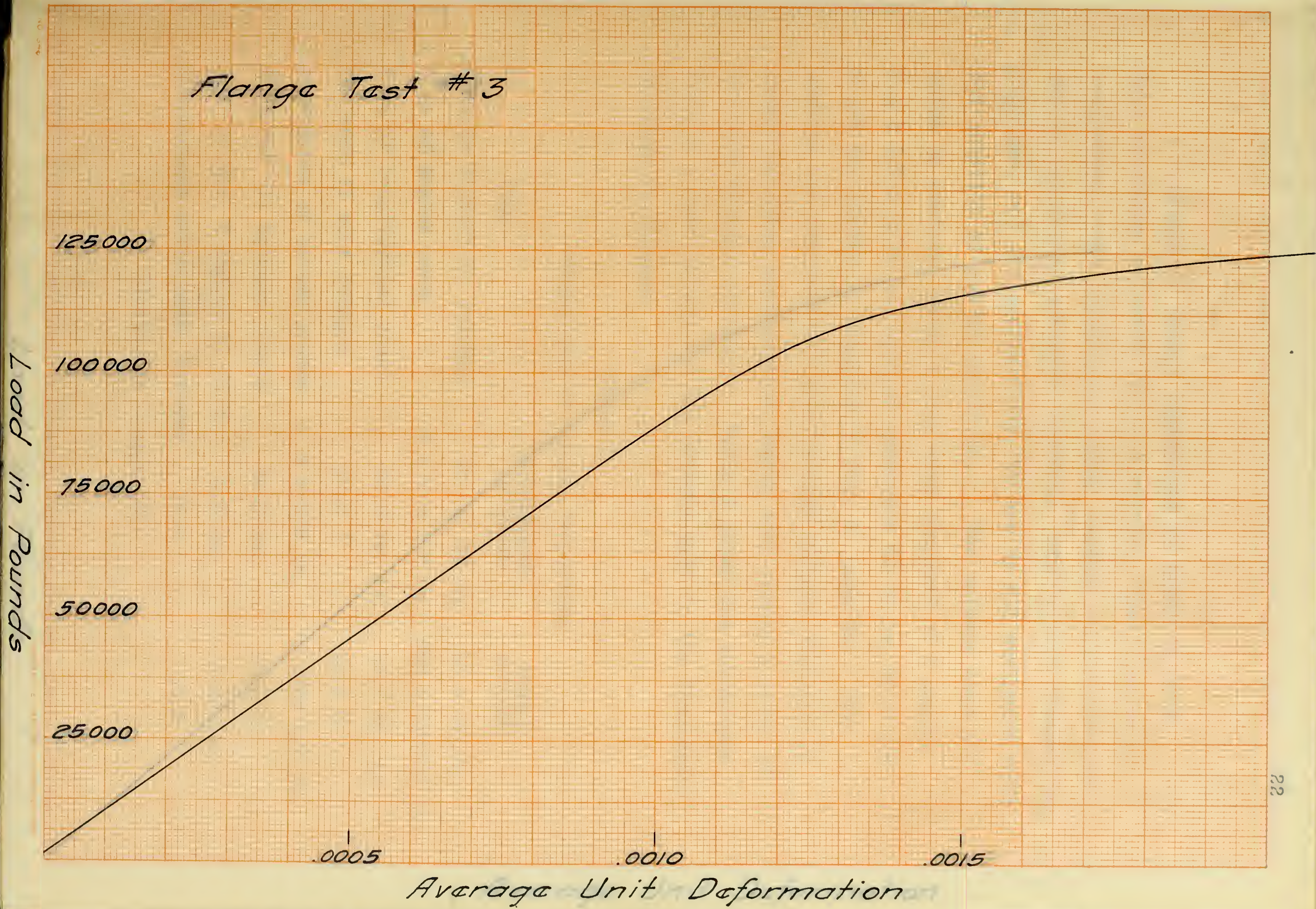

Flange Test #1.
100000
75000
50000
25000
.0005
.0010
.0015
Average Unit Deformation
Load in Pounds



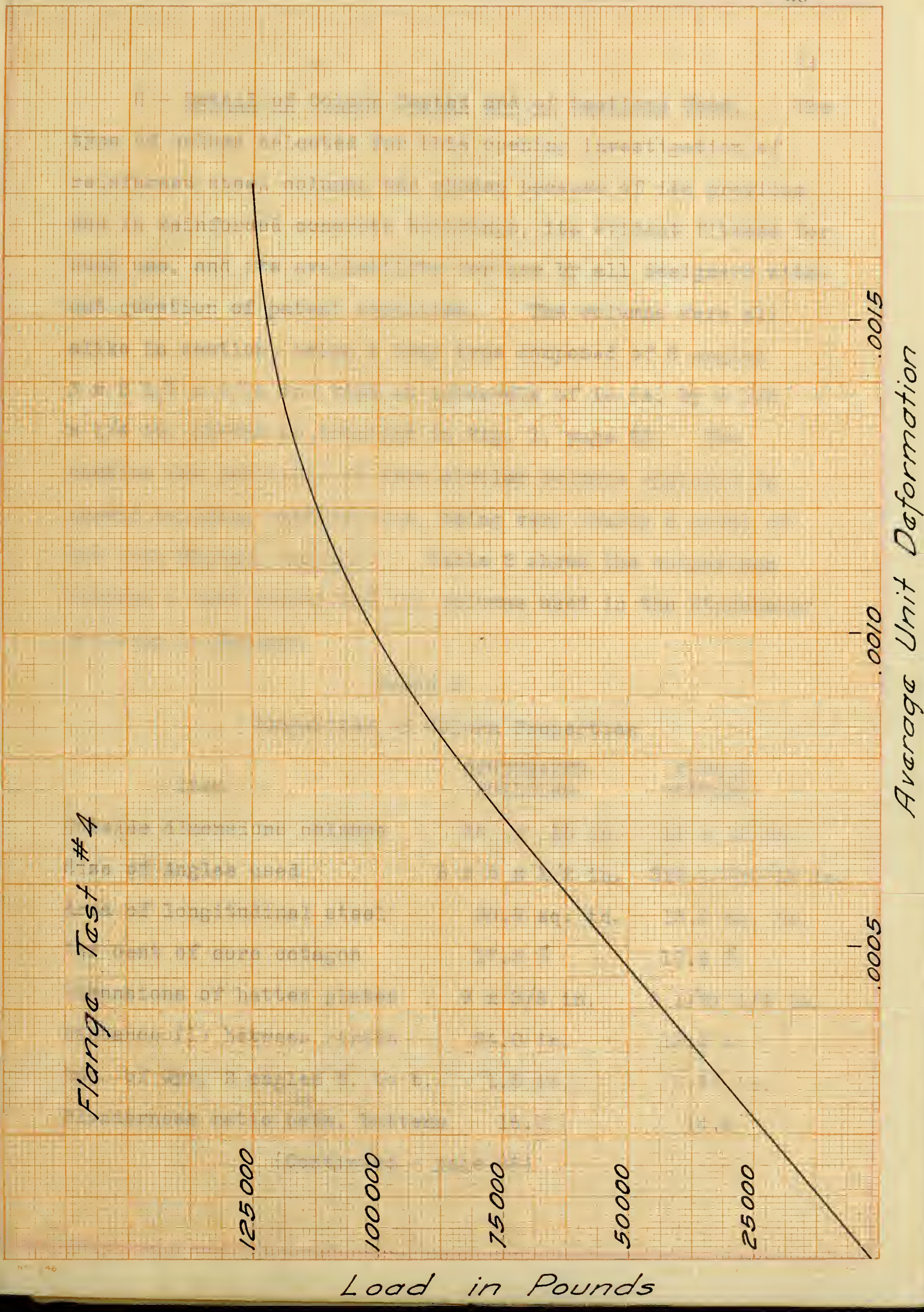
Flange Test # 2
100000
75000
50000
25000
.0005
.0010
.0015
Average Unit Deformation
Load in Pounds



Flange Test
Load in Pounds
Average Unit Deformation


Flange Test # 3
125000
100000
75000
50000
25000
Load in Pounds
.0005
.0010
.0015
Average Unit Deformation


Flange Test #3
125000
100000
75000
50000
25000
Load in Pounds
.0015
.0010
.0005
Average Unit Deformation


Flange Test #4
125000
100000
75000
50000
25000
.0005
.0010
.0015
Average Unit Deformation
Load in Pounds


Flange Test #4
125000
100000
75000
50000
25000
Load in Pounds
.0015
.0010
.0005
Average Unit Deformation

8 – Detail of Column Tested and of Sections Used. The type of column selected for this opening investigation of reinforced steel columns was chosen because of its previous use in reinforced concrete buildings, its evident fitness for such use, and its availability for use by all designers without question of patent royalties. The columns were all alike in section, being a Gray type composed of 8 angles 3 x 2 1/2 x 5/16 in. tied at intervals of 16 in. by 5 1/2 x 1/4 in. plates as detailed in Fig. 1, page 25. The section was proportioned from similar columns employed in recent building construction, being very nearly a model of two late Chicago designs. Table 2 shows the comparison between a test column and the columns used in the Studebaker Building in Chicago.

Table 2

Comparison of Column Properties

Item	STUDEBAKER BUILDING	THESIS COLUMNS
Outside dimensions columns	16 x 16 in.	12 x 12 in.
Size of Angles used	6 x 4 x 1/2 in.	3x2 1/2x5/16 in.
Area of longitudinal steel	38.8 sq. in.	13.0 sq. in.
Per cent of core octagon	17.3 %	10.8 %
Dimensions of batten plates	9 x 3/8 in.	5 1/2x 1/4 in.
Distance (l) between rivets	24.0 in.	13.5 in.
Rad. of Gyr. 2 angles b. to b.	1.5 in.	0.95 in.
Slenderness ratio betw. battens	16.0	14.2

(Continued – page 26)

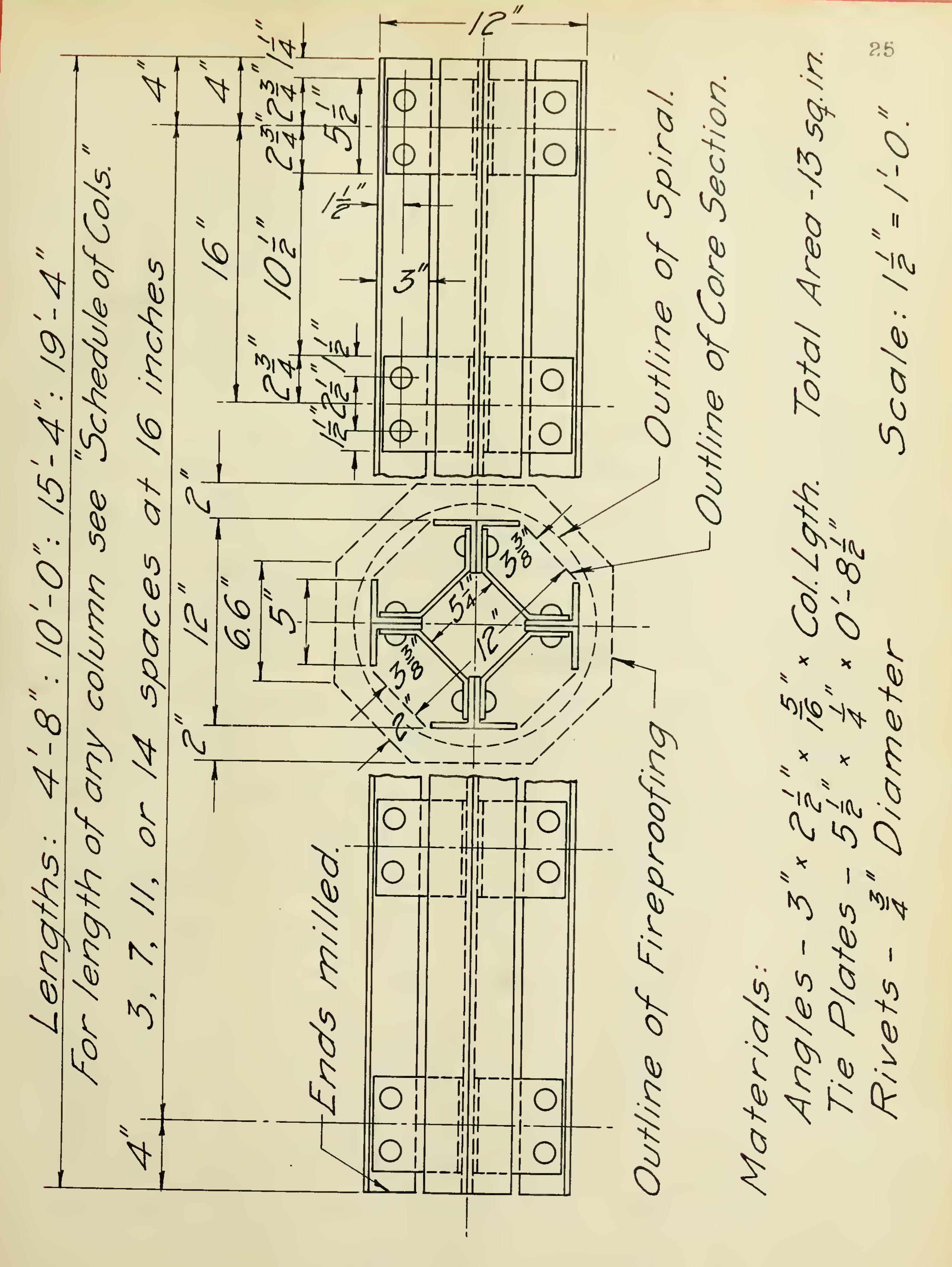


FIG. 1 – DETAILS OF TEST COLUMNS

Table 2 (Continued)

Item	STUDEBAKER BUILDING	THESIS COLUMNS
Length of column considered clear	12 ft.	10 ft.
Radius of gyration of column	4.6 in.	3.9 in.
Slenderness ratio for column	31.3	30.8
Ratio of length to core diameter	9.0	10.0

The columns were fabricated at the North Works of the Illinois Steel Company in Chicago and shipped to the University during the summer of 1910. The ends of the columns were milled as is common in building construction and the entire fabrication was handled on a strictly commercial basis, no especial care being taken and the resulting columns being in no sense excéptional. The ends, although milled, were far from presenting a true bearing and in the case of the shortest columns had to be somewhat modified in the laboratory (by filing, etc.) before any satisfactory test was possible. Ten columns were tested practically as received and these are noted as "plain steel" in the schedule given in the summary sheet (folded insert). For studying the effect of filling the column with concrete the standard section used was that noted as "core section" in the table and shown in outline in Fig. 1, page 25, in which the concrete was entirely confined within the angles forming a solid octagon five inches on a side. This section was adopted as a standard because it was believed wisest to obtain the larger part of the data on the actual effective section which is considered in design, rather than

upon the fireproofed column. For comparative purposes, however, three columns were made with two inches of fire protection outside the steel, and the action of this fire resisting shell under load was studied. The effect of various mixtures of concrete upon the strength of the core section was sought by the testing of two columns each of 1:1:2 and 1:3:6 mixtures and comparing them with like columns of the standard 1:2:4 mixture. In six columns the steel was enclosed within a wire spiral and filled with concrete to the face of the spiral. The percentages of spiral reinforcement used were 0.75 % and 1.0 %. The outlines and dimensions of the various sections used are shown in Fig. 1. The percentage of the structural steel to the whole column area varies with the different sections, being 10.8 % for the Core Section, 6.1 % for the fireproofed section, and 8.5 % for the spiraled columns. The summary sheet shows the number of columns of each type of section which were submitted to test.

9 - Making of Columns. The forms for the core section were extremely simple inasmuch as the concrete was confined to the octagon determined by the edges of the steel flanges. Four planks of the correct length were placed in a vertical directly against and closing the openings between the flanges and were held in position by means of yokes. For the fire-proofed columns octagonal forms of wood were built giving the required two inches of clearance over all faces. For the spiraled sections metal forms were placed directly against

Table 3

Schedule of Columns

Col. No.	Length	Mixture	Age Days	Description
8902	2' – 0"	. . .	. .	Plain Steel
8905 06	4' – 8"	. . .	. .	Plain Steel
8907 08	4' – 8"	1:2:4	60 59	Core Section
8910 11	10' – 0"	. . .	. .	Plain Steel
8912 13	10' – 0"	1:2:4	60 62	Core Section
8914	10' – 0"	. . .	. .	Plain Steel
8915 16	15' – 4"	. . .	. .	Plain Steel
8917 18	15' – 4"	1:2:4	61 59	Core Section
8920 21	19' – 4"	. . .	. .	Plain Steel
8922 23	19' – 4"	1:2:4	60 60	Core Section
8925 26	10' – 0"	1:1:2	61 60	Core Section
8927 28	10' – 0"	1:3:6	59 60	Core Section
8929 30 31	10' – 0"	1:2:4	60 60 60	2" Fireproofing
8933 34 35 36	10' – 0"	1:2:4	60 59 59 60	3/4 % Spiral
8937 38	10' – 0"	1:2:4	60 59	1 % Spiral

Further data will be found in general summary sheet (folded)

An Investigation of Rein[forcement]

[Rein]cement: Str. 3"-16" Cylinders 60 da.	Lateral Steel: Size	Lateral Steel: Pitch	Lateral Steel: %	Total Load on Column	Ultimate Load: Load Carried by Steel	Ultimate Load: Load Carried by Reinforc.	Unit: Ultimate Unit Load for Column
#/□"	φ"	"	%	Lbs.	Lbs.	Lbs.	#/□"
				487300			37500
				440200			33860
				449000			34540
1350				577000	444600	132400	4808
1490				602000	444600	157400	5017
				410400			31570
				425600			32740
1350				510000	418000	92000	4250
1490				584700	418000	166000	4870
				424000			31800
				368000			28310
				376000			28920
1142				468500	372000	96500	3905
1264				532200	372000	160200	4435
				374500			28800
				345000			26550
976				491400	359600	131800	4055
1155				495500	359600	135900	4130
2426				636000	418000	218000	5300
2525				655000	418000	237000	5460
705				516000	418000	98000	4300
663				530500	418000	112000	4420
1372				600000+	Not Broken.		2820 +
1336				630700	418000	212700	2990
1280				635700	418000	217700	2960
1164	0.25	2	0.75	600000+	Not Broken.		
1342	"	2	"	860000	418000	432000	5610
1330	"	2	"	830000+	Not Broken.		
1109	"	2	"	600000+	Not Broken.		
1465	"	1.5	1.0	830000+	Not Broken.		
1120	"	1.5	1.0	827000+	Not Broken.		
				103400			31800
				110800			34090
				128500			39500
				130000			40000

Thesis of Arthur R. Lord — 1911

An Investigation of Reinforced Steel Columns

Summary of Reduced Test Data

Column No.	Description	Length	L ÷ r	Area Gross Section	Reinforcement: Concrete: In Column: Area	Reinforcement: Concrete: In Column: %	Reinforcement: Concrete: Strength 6" Cubes: 60 da.	Reinforcement: Concrete: Strength 6" Cubes: 90 da.	Str. 8"·16" Cylinders: 60 da.	Lateral Steel: Size	Lateral Steel: Pitch	Lateral Steel: %	Ultimate Load: Total Load on Column	Ultimate Load: Load Carried by Steel	Ultimate Load: Load Carried by Reinforc.	Ultimate Unit Load for Column
UNIT		FT IN.		□"	□"	%	#/□"	#/□"	#/□"	"φ	"	%	Lbs.	Lbs.	Lbs.	#/□"
8902	Plain Steel	2 - 0	6.1	13									487300			37500
8905	do	4 - 8	14.4	13									440200			33860
06	do	do	14.4	13									449000			34540
07	Core Section	do.		120	107	89.2	1795		1350				577000	444600	132400	4808
08	do	do		120	107	"	2436	2651	1490				602000	444600	157400	5017
8910	Plain Steel	10 - 0	30.8	13									410400			31570
11	do	do	30.8	13									425600			32740
12	Core Section	do		120	107	"	1795		1350				510000	418000	92000	4250
13	do.	do		120	107	"	2436	2651	1490				584700	418000	166000	4870
14	Plain Steel	do.	30.8	13									424000			31800
8915	do.	15 - 4	47.2	13									368000			28310
16	do.	do.	47.2	13									376000			28920
17	Core Section	do		120	107	"	1422		1142				468500	372000	96500	3905
18	do.	do.		120	107	"	2149	2600	1264				532200	372000	160200	4435
8920	Plain Steel	19 - 4	59.5	13									374200			28800
21	do.	do.	59.5	13									345000			26550
22	Core Section	do.		120	107	"	1324		976				491400	359600	131800	4095
23	do.	do.		120	107	"	1972	2638	1155				495500	359600	135900	4130
8925	do.	10 - 0		120	107	"	2972		2426				636000	418000	218000	5300
26	do.	do		120	107	"	3286	4000	2525				655000	418000	237000	5460
8927	do	do		120	107	"	1326	1800	705				516000	418000	98000	4300
28	do.	do.		120	107	"	1434	1745	663				530500	418000	112000	4420
8929	2" Fireproofing	do		213	200	93.9	1760	2582	1372				600000+	Not Broken.		2820 +
30	do.	do		213	200	"	2020	2740	1336				630700	418000	212700	2990
31	do.	do.		213	200	"	1673	2260	1280				635700	418000	217700	2960
8933	Spiral Section	do		153	140	91.5	1438	2000	1164	0.25	2	0.75	600000+	Not Broken.		
34	do	do.		153	140	"	2070	2355	1342	"	2	"	860000	418000	432000	5610
35	do	do		153	140	"	2168	2720	1330	"	2	"	830000+	Not Broken.		
36	do.	do.		153	140	"	1785	2180	1109	"	2	"	600000+	Not Broken.		
8937	do.	do		153	140	"	1624	2900+	1465	"	1.5	1.0	830000+	Not Broken.		
38	do	do.		153	140	"	1985	2580	1120	"	1.5	1.0	827000+	Not Broken.		
Flg #1	Single Flange	7 - 10	100	3.25									103400			31800
#2	do.	7 - 11	100	3.25									110800			34090
Flg #3	do	2 - 8	35	3.25									128200			39500
#4	do.	2 - 8	35	3.25									130000			40000

Column No.	Unit Load: Ult. Unit Load for Reinforcement: Value fr. Curves	% of Cube	% of Cyl.	Secant Modulus of Elasticity at Unit Deformation of .0004: Steel	.0004: Concr.	.0004: Ratio	.0007: Steel	.0007: Concr.	.0007: Ratio	.0010: Steel	.0010: Concr.	.0010: Ratio	Near Ultimate Deformation: Steel	Near Ultimate Deformation: Conc.	Near Ultimate Deformation: Ratio	Value Unit Deformation Last Reading	Initial Mod. of Elasticity: From Conc. 8"×16"	Initial Mod. of Elasticity: Item of Cyl.'r Curves	Column No.
UNIT	#/□"	%	%	1,000,000 #/□"			1,000,000 #/□"			1,000,000 #/□"			1,000,000 #/□"				1,000,000 #/□"		
8902				34.25			32.55			30.60			26.55			.0017			8902
8905				31.35			28.70			26.40			22.80			.0016			8905
06				31.15			29.60			27.30			25.10			.0014			06
07	1237	69	92	31.25	1.073	29	29.15	0.800	35	26.85	0.700	38	23.85	0.640	37	.0018	2.00	2.00	07
08	1471	60	99	31.25	1.330	23	29.15	1.200	24	26.85	1.110	24	23.85	1.030	23	.0015	2.08	1.67	08
8910				30.60			28.30			26.05			20.30			.0015			8910
11				29.80			27.80			25.75			22.30			.0014			11
12	860	48	64	30.20	1.216	25	28.05	0.920	30	25.90	0.730	35	21.30	0.567	37	.0017	2.00	2.00	12
13	1558	64	104	30.20	1.463	21	28.05	1.320	21	25.90	1.190	22	21.30	1.030	21	.0014	2.08	2.30	13
14				31.30			29.45			27.40			24.25			.0013			14
8915				29.80			26.90			24.45			23.00			.0012			8915
16				30.80			27.50			24.60			20.50			.0014			16
17	902	63	79	30.30	0.864	35	27.20	0.750	36	24.50	0.675	36	21.80	0.635	34	.0012	1.85	1.46	17
18	1497	70	118	30.30	0.982	31	27.20	1.070	25	24.50	1.030	24	21.80	0.900	24	.0013	1.55	1.79	18
8920				28.90			26.00			23.55			22.45			.0012			8920
21				30.00			26.60			23.75			23.15			.0011			21
22	1232	93	126	29.45	0.942	31	26.30	0.840	31	23.65	0.740	32	22.80	0.740	31	.0010	1.30	1.81	22
23	1270	65	112	29.45	1.239	24	26.30	1.130	23	23.65	1.110	21	22.80	1.050	22	.0012	1.35	1.87	23
8925	2037	68	84	30.20	1.986	15	28.00	1.720	16	25.90	1.610	16	21.30	1.400	15	.0016	2.84	2.53	8925
26	2215	67	88	30.20	1.636	18	28.00	1.470	19	25.90	1.430	18	21.30	1.360	16	.0020	3.15	1.85	26
8927	916	69	130	30.20	0.795	38	28.00	0.630	44	25.90	0.520	45	21.30	0.582	37	.0016	1.25	1.30	8927
28	1051	73		30.20	0.748	40	28.00	0.630	44	25.90	0.600	43	21.30	0.600	35	.0015	1.25	1.06	28
8929				30.20	1.275	24	28.00	1.092	26	25.90	1.000	26	21.30	0.930	23	.0012	2.00	1.60	8929
30	1063	53	80	30.20	0.763	40	28.00	0.730	38	25.90	0.690	37	21.30	0.663	32	.0018	1.67	1.00	30
31	1088	65	85	30.20	1.223	25	28.00	0.957	29	25.90	0.780	33	21.30	0.672	32	.0018	2.00	1.85	31
8933				30.20	1.340	23	28.00	1.160	24	25.90	1.110	23					1.37	1.67	8933
34	3080			30.20	1.410	21	28.00	1.060	26	25.90	0.886	29				.0054	1.25	2.00	34
35				30.20	1.520	20	28.00	1.340	21	25.90	1.290	20				.0046	2.00	2.00	35
36				30.20	1.360	22	28.00	1.140	25	25.90	1.060	24					2.00	1.70	36
8937				30.20	1.550	19	28.00	1.330	21	25.90	1.220	21				.0034	2.00	2.00	37
38				30.20	1.300	23	28.00	1.180	24	25.90	1.110	23				.0038	2.00	1.80	38
Flg #1				30.00			30.00			29.10			29.00			.0010			Flg #1
#2				30.40			30.20			29.10			28.30			.0012			#2
Flg #3				27.40			27.40			26.90			24.00			.0015			Flg #3
#4				32.40			31.90			30.50			22.60			.0017			#4

the wire spiral and held in place by bands as shown in Fig. 1 Bulletin No. 20, University of Illinois Engineering Experiment Station.

The concrete was mixed by hand on the floor of the preparation laboratory by men experienced in practical work as well as in the preparation of test specimens. The materials were proportioned by weight and the water was also weighed as added in most cases. The sand and cement were first turned five or six times and then three more dry turnings were given the mass after the stone was added. Water was added in sufficient quantity to produce a distinctly wet mixture which would run rapidly from the shovel. The concrete was placed in the column from the top a bucketful at a time and thoroly worked around the sides and in the center by means of a pole. As an aid to securing uniform concrete successive bucketfuls were taken from different portions of the pile. The forms were filled practically level with the top of the steel.

Previous to pouring the steel columns were placed in a vertical position upon a 14 x 14 x 1 in. cast iron bearing plate (on which it was later tested) and the forms placed in position. About a day after pouring, when the concrete in the column had had time to shrink, the top of the column was prepared for testing by setting another bearing plate upon it using a neat cement mortar for this purpose. Care was taken to have this plate resting squarely upon the steel and to keep the layer of mortar between the plate and the steel as thin as possible. A thin film of cement was necessarily

present, but in most cases it was extremely thin and in no case did it exceed 1/8 inch in thickness. In pouring the spiral section it was found that the concrete would not flow into the small space between the back of the flanges and the spiral and it was necessary to use a sand mortar to grout in this space. Somewhat the same difficulty was experienced in the fireproofed columns, but in these concrete was used resulting in a badly pitted surface over the flanges in many spots. This effect may be noticed in the photographs of the fireproofed columns.

10 - Auxiliary Test Specimens. From each batch of concrete used three six-inch cubes and one 8 x 16 in. cylinder were made from which to determine the properties of the concrete in the columns. These were packed in damp sand until a few days before the column was to be tested when they were removed to the testing laboratory and two faces prepared for the test by the addition of a thin coat of plaster of paris. It was originally intended to test all these specimens at the same date as the corresponding column, but it was found that an unexpected variation was shown in the cube tests, the strength being lower than would be expected. For this reason one cube from each of the later batches was reserved for test at 90 days in order to know whether the concrete was poor or merely lacked curing. The results of the cube and cylinder tests are given in the general summary sheet (folded insert) These auxiliary tests show quite conclusively that another and undesired

variable - namely the strength of the concrete at the time of testing - has been introduced into the investigation. Its effect is not negligible and will be treated more in detail in a later paragraph.

11 - Storage and Handling. The columns were kept where made in the preparation laboratory. Forms were removed at the end of a week or ten days and from that time on the columns were occasionally sprinkled. The temperature records show a variation between 60° and 70° Fahrenheit, but it seems probable that larger variations may have occurred in some parts of the building. Some of the longer columns were built near steam pipes which were attached to the upper portion of one wall of the laboratory, and these columns were certainly subjected to higher temperature than the other columns and their tops were more highly heated than their bases. The temperature of the columns was also different from that of the damp sand in which the cubes and cylinders were stored. It seems probable that the columns dried out much more and had attained a higher percentage of their final strength (insofar as the concrete affects that strength) than the cube tests would indicate.

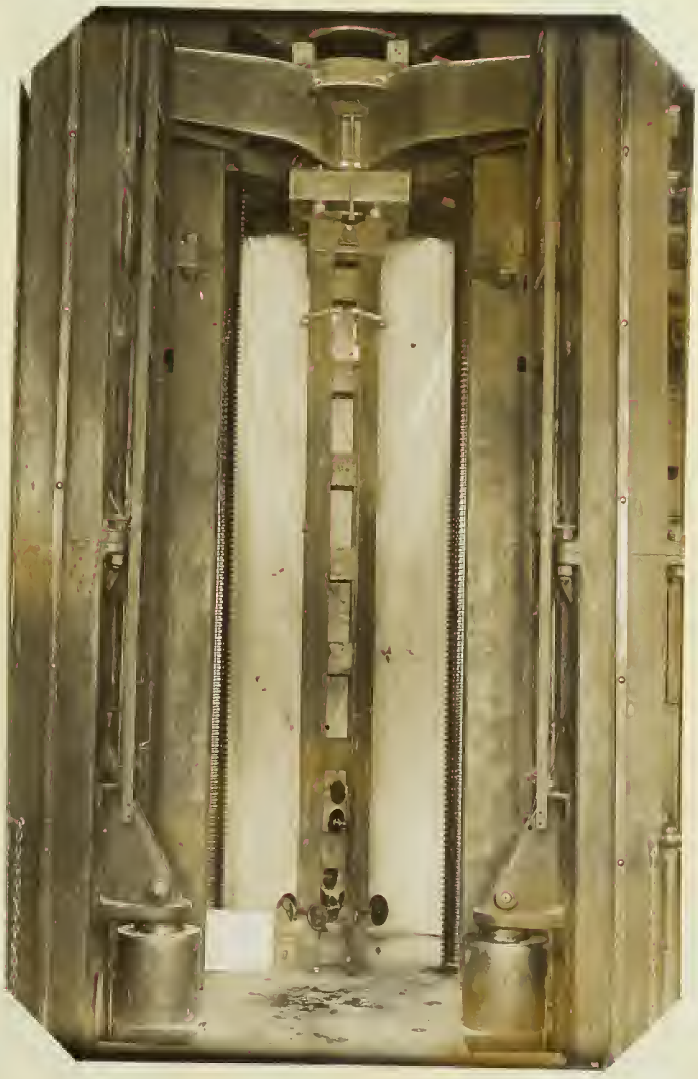
Before removing the columns to the testing laboratory the bearing plates at top and bottom were connected by rods to prevent displacement and these rods were not removed until the column was in its final position in the testing machine.

12 - Attachment and Description of Apparatus. The apparatus used and the method of attachment are well shown in the photographs of columns ready for testing. One of

COLUMN NO. 8 9 0 2

SHOWING ARRANGEMENT OF APPARATUS

these appears as the frontispiece of this thesis. The photograph of the small column on page 32 brings out the detail especially well. On the steel deformations were measured by tapping holes in the flanges to receive the shaft of a wire-wound dial at the bottom and to receive a bolt at the top - from this bolt a wire was suspended, wrapped once around the drum of the dial, and weighted with a nut at its lower end. Thus any deformation occurring in the gauge length between the bolt and the dial was registered by the movement of the pointer of the dial. Where measurements were desired on the concrete faces, specially prepared plugs were inserted in the column during the pouring, and these were tapped to receive the bolts and dials. The wire suspended from the upper bolts was in general one inch from the face of the column, and the accuracy of the observations depends on the conservation of pla~~in~~ne section in the column as a whole. The results obtained indicated that this condition was not fully satisfied in the case of the plain columns, and a slight error is shown, being especially noticeable in the computed moduli of elasticity for the steel. A different arrangement of instruments, shown in the photograph on page 34, was used on the flange tests and on column No. 8914 and indicates that the error due to the arrangement most used was not great. The gauge lengths used varied with the lengths of the specimens, being roughly proportional to those lengths as shown in the following table. The dials could be read to an indicated movement of .0002



COLUMN NO. 3 9 1 4

CONSERVATION OF PLANE SECTION

inches. This being constant for all gauge lengths the least unit deformàtion possible of measurement depended on the length of the gauged distance. The least unit-deformation obtainable is indicated for each gauge length in the table below.

Table 4

Gauge Lengths and Least Unit Deformations

Column Length	Gauge Length	Least Unit-Deformation
2' - 0"	10"	.00002 (in. per in.)
4' - 8"	40" (varied)	.000005
10' - 0"	100"	.000002
15' - 4"	150"	.00000133
19' - 4"	200"	.000001

In the case of the plain steel columns 10 feet or more in length, four measurements of deformation were taken over the gauge lengths indicated above, and six other measurements of deformation were taken over gauge lengths about one-third as long. These shorter gauged lengths were located at different portions of the columns as shown in the photographs of the plain steel columns.

13 - Methods of Testing. In the case of the plain steel columns the specimen was placed with its lower end bearing directly on the bed of the testing machine and centered with respect to the screws. The upper head of the machine was then lowered until the suspended spherical block rested on top of the column. This block was then centered on the column and the deformation measuring instruments were attached as

described in the preceding paragraph. The ends of the columns were milled in the shops but in many cases it was necessary to resort to shimming or light filing to secure a satisfactory bearing of the block on the column. In the case of the reinforced columns the bearing plates were first scraped clean and then the column was carefully centered in the machine. When in its final position the rods connecting the plates were removed and the spherical block lowered into position as noted above for plain steel columns. The instruments for measuring deformations having been attached an initial reading of the dials was taken with no load on the column except the weight of the spherical bearing block. The upper head of the testing machine was then brought to bear on the block and run down at the slowest speed of 0.05 inches per minute until a load of 25 000 pounds was registered. The machine was stopped at this load and after an interval of from 30 to 45 seconds the dials were read. In like manner the load was increased by increments of 25 000 pounds each, alternating with readings of the dials, until the column crushed or the capacity of the testing machine was reached. At a load of 50 000 pounds, after the deformations had been recorded, the special wedges (described in paragraph "Condition of End Restraint" following) were adjusted at the top of the column. With the first five columns tested (Nos. 8905, 8906, 8910, 8915, and 8920) these wedges were not used. Many columns were tested further after the ultimate or maximum load was passed, the result being marked bending. This was done in order that the critical section might be definitely

determined and the nature of the final failure observed. Three conditions mentioned incidentally above merit more complete discussion:

a – Speed of Testing Machine. The question of the speed at which the testing machine shall be run seems to the writer to be more important than is sometimes held. Frequently no statement of this speed is offered at the time of publication of the data. It seems evident, however, that a column will carry a higher maximum load if the load be applied rapidly than it would under a slow application. This fact is very generally appreciated among testing engineers and does not lack for experimental verification. It follows that in a series of tests the rate of application of load should be very slow unless a misleading showing of strength is desired, for in actual constructions the load is not momentarily applied and then partially released owing to the resulting deformation of the column, but is a dead weight and tends to follow up any shortening, a condition of much greater severity in its effects on column strength. The load in a testing machine is applied by the propulsion of the head downward upon the column. Under high unit loads the specimen tends to shorten for some time after the motion of the head is arrested. The result is that the actual load on the column may fall off considerably since the head cannot follow up and maintain it. Thus it appears that the maximum load as indicated by the testing machine is always higher (for given conditions as to bearing and restraint) than could be

carried by the column as an actual dead weight. It is evident, further, that for truly comparable tests the rate of deformation (or of increase in stress) should be constant. This would mean that the speed of the testing machine should vary directly with the length of the specimen. With our present testing machines it is not feasible to attain this latter object, and in the tests described in this thesis the slowest speed of 0.05 inches per minute was used in all cases, which satisfies the first condition as closely as possible but does not attempt to satisfy the second. The load was not held at its full value for any considerable time, but was allowed to decrease when the deformation of the specimen permitted. The instruments were read starting about 30 seconds after the indicated load had been momentarily applied, and the load on the column at the time of reading was sometimes less than the full indicated load.

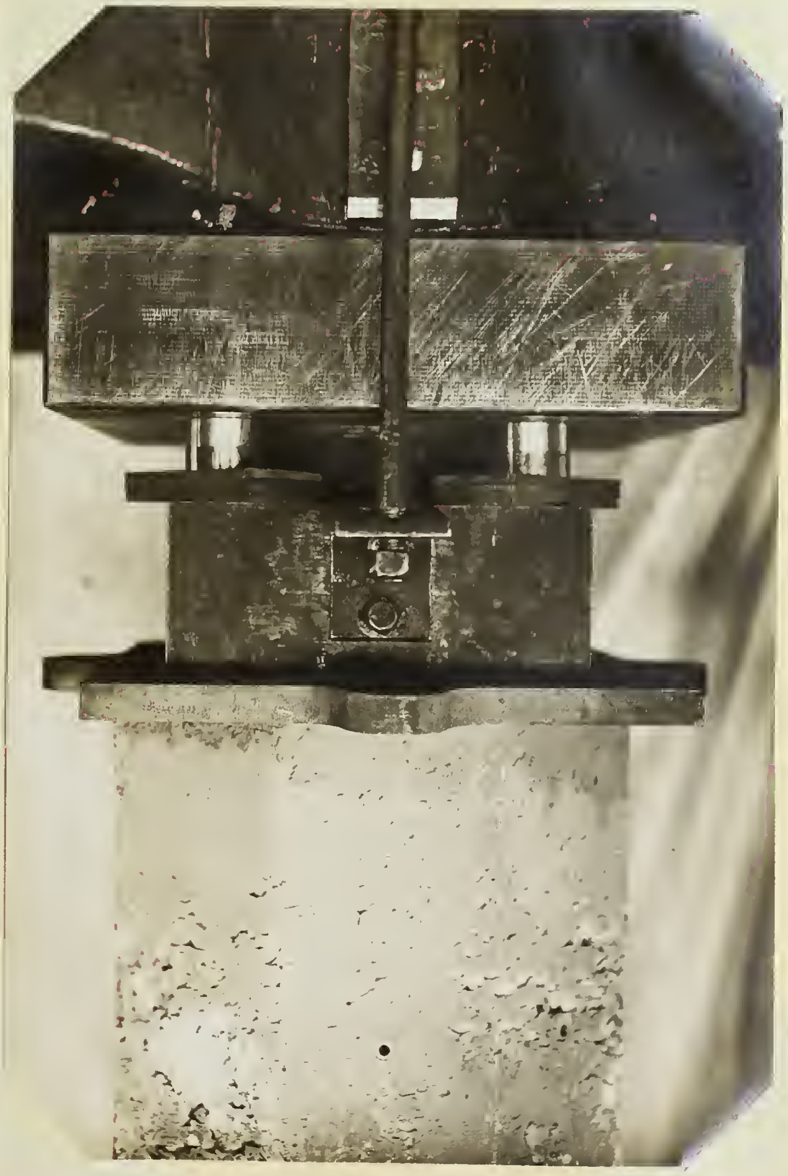
b - Condition of End Bearings. While the nature of the loading device in a testing machine tends to give abnormally high indicated maximum loads, there are other conditions which act to reduce the indicated carrying capacity in the laboratory test. The first in importance, as it is the hardest to overcome, is the condition of the bearing surfaces at the ends of the column. Where in a building the column receives its load by increments from story to story (especially is this true of reinforced steel columns), in a testing machinethe load is applied directly upon the end sections. If the load is to be uniformly distributed,

the bearings of the ends on the loading blocks of the machine must be perfect. The columns prepared for this thesis investigation were milled at both ends, as is the practise in construction, but the end conditions were far from perfect. Careful shimming and filing resulted in much improvement but it must be recognized that the load was not uniformly distributed over the section and that some flanges were more highly stressed than others even after the most careful treatment in this crude way. The record of observed deformations on the various gauge lengths (see Part VI, Logs of Tests) show this effect very plainly. When the load is unevenly distributed over the section it is evident that the tendency toward bending is aggravated and the column is sensibly weakened. The effect of poor end conditions is more serious in short columns than in long ones - in the 19 ft. 4 in. columns its effect is probably negligible. In studying the effect of length on column strength the effect of end conditions must be constantly borne in mind. Columns Nos. 8902 and 8914 had the best end conditions of any tested in this series - in the 2 ft. column (8902) a test was found impractical with the ends as received and they were finally carefully turned in a lathe.

c - Condition of End Restraint. A second important condition as affecting the indicated carrying ~~carrying~~ capacity of the column in the testing machine, is the fixidity of the ends of the column. The lower end bearing on the heavy unyielding base of the testing machine is evidently quite firmly restrained until serious bending occurs in the column.

The upper end, however, must be loaded by means of a spherical bearing block in order to secure even a fair distribution of load over the section, and this block is free to turn as the column develops a tendency to bend, except for the friction developed in the block itself. In the first five plain steel columns tested (Nos. 8905, 8906, 8910, 8915, and 8920) no effort was made to prevent this accommodation and the column was in the condition of one end partially fixed and one end partly hinged or free. Neither the fixidity or the freedom can be considered as in any sense absolute, but they must be taken simply as relative terms as used here. In all the later tests special wedges and angle blocks were driven under the upper head at a point in the test (usually at 50 000 pounds load) when the spherical block had had opportunity to adjust the bearing satisfactorily but before any appreciable tendency to bending developed in the column, and in this manner the spherical bearing block was restrained from any further motion becoming in effect a rigid loading block like the base. The photograph on page 41 shows the apparatus in position during a test. That the apparatus accomplished the desired purpose is evidenced by the nature of the failure in bending which occurred almost invariably at or below the center of length of the column. In all cases where the strength of the column with and without the wedges could be compared, save only the 19 ft. 4 in. length, the strength developed was greater with the wedges.

It should be noted in this connection that the conditions



VIEW OF UPPER BEARING BLOCK

SHOWING FIXIDITY OF TOP OF COLUMN

of restraint in the test specimen, even when the upper block has been fixed by means of wedges, are not so satisfactory as is the case with a column incorporated in a building. In the building there is present at the two ends a tensile resistance to bending, which is totally lacking in the test specimen. Hence it must be considered that the slope of the neutral axis is maintained more nearly vertical in building construction than is the case in test columns, resulting in a higher strength in the more nearly fixed-ended building column.

It seems to the writer that the conditions acting to lower the indicated carrying capacity of the test columns are more influential than are the conditions tending to raise it, and that it is to be expected that the a column loaded as an integral part of a building will carry a higher load than will the same column tested as were those in this thesis investigation.

III

EXPERIMENTAL DATA

AND DISCUSSION

PLAIN STEEL COLUMN

READY FOR THE TEST

III EXPERIMENTAL DATA AND DISCUSSION

A PLAIN STEEL COLUMNS – EFFECT OF LENGTH

14 – Phenomena of the Tests and of Failure. The plain steel columns showed test phenomena which were consistent and uniform. The logs of the tests will be found in Part VI at the end of this thesis. At a load of from 225 000 to 250 000 pounds cracking sounds were heard and these continued intermittently thruout the remainder of the test. The time required to add the 25 000 lb. increment to the load gradually became longer until at the ultimate load the beam floated within a range of a thousand pounds for a period of from ten to fifteen minutes, and the yielding from that time on to final collapse was exceedingly gradual. The column seemed to possess to a high degree the desirable qualities of toughness and slow failure. At maximum load very little bending (or increase in bending) was shown by the deformation readings, and none was visible to the eye except in the longer columns. After the maximum load was reached and the machine head was run down to complete the failure of the column (generally at faster speed), bending developed very gradually and a slight twisting action was noticed in the

COLUMN NO. 8 9 0 5

FAILURE AT LOWER END

COLUMN NO. 8 9 0 6

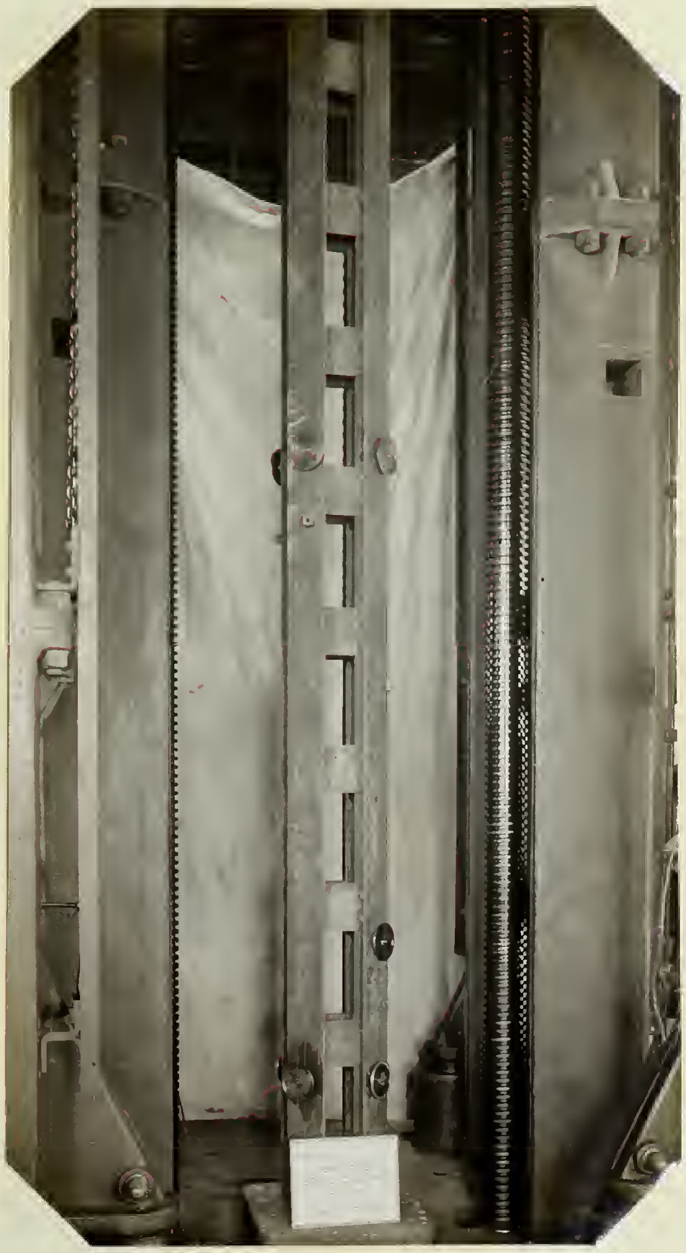
SYMMETRICAL FAILURE

COLUMN NO. 8 9 1 0

TESTED TO DESTRUCTION

COLUMN NO. 8 9 1 6

SYMMETRICAL FAILURE



COLUMN NO. 8 9 2 0

INSTRUMENTS ATTACHED

COLUMN NO. 8 9 2 0

SYMMETRICAL FAILURE

COLUMN NO. 8 9 2 1

FAILURE ABOVE CENTER

columns of ten feet or more length. The photographs on pages 44 to 52 give a good idea of the nature of the final failure in bending, and show it to have been in general quite symmetrical about the center.

15 – Stress-deformation Relations. The nature of the load-deformation diagrams (pp 124 to 129) for the plain steel columns is a matter of interest, although, as noted elsewhere, these curves are subject to slight errors owing to the fact that the arrangement of instruments presupposed the conservation of plain section in the column during the test, a condition which seems not to have been fully satisfied. The bending in the load-deformation curves which occurs at low loads is undoubtedly due largely to this cause, as the curve for column No. 8914, obtained with a different arrangement of instruments, shows a straight line up to 175 000 pounds load. In the general summary (folded insert) the secant moduli of elasticity, calculated from the curves, are given for unit deformations of .0004, .0007, and .0010, as well as for a point near the ultimate load. From these computations it would appear that the modulus for low deformations may be considered as lying between 30 000 000 and 31 500 000 pounds per square inch. It is also apparent that the modulus decreases as the length of the column increases. The load-deformation curves do not show any decided yield point and indicate well the gradual nature of the failure.

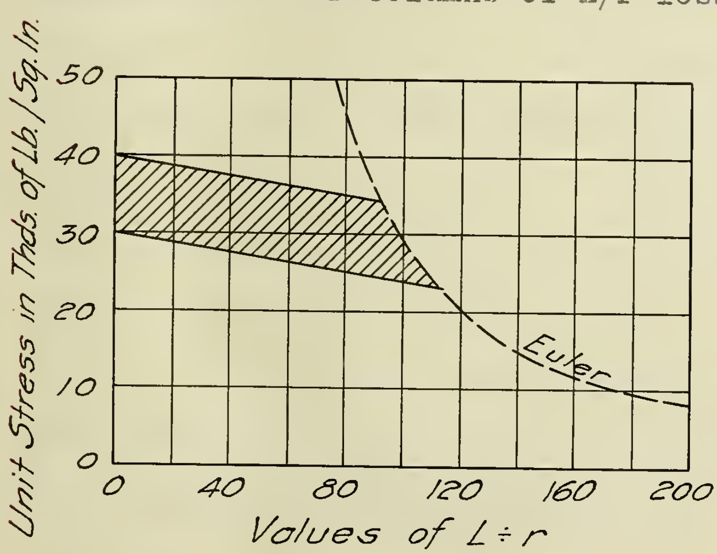
16 – Effect of Test Conditions. In the plain steel columns, as was to be expected, the effect of imperfect end bearings was most strongly shown. The tables on pages

184 to 199 give the observed deformations over the various flanges and show quite wide variations. One column, No. 8915, was considerably weakened by the fact that one angle of one flange lacked a full quarter of an inch of bearing on the top block. Column No. 8914, whose ends were filed quite carefully to a fairly true surface, showed a marked increase in stiffness and a higher bend in the load-deformation curve than did its mates Nos. 8910 and 8911, which were tested with their ends only slightly retouched. The diagram on page 127 shows a comparison of the load-deformation curves for the four flanges of column No. 8914. On page 125 is shown the same curves for columns Nos. 8910 and 8911. By placing one diagram over the other a very good idea may be obtained of the difference in stiffness due to end conditions. Column No. 8914 carried more load than No. 8910 but no more than No. 8911.

The fact that the columns finally failed in bending at points either at or below the middle has been mentioned before. It is sufficient evidence that the rigidity was as great at the upper block (when wedges were used) as at the base. Column No. 8910 (not wedged) carried 410 000 pounds, column No. 8911 (wedged) carried 425 000 pounds and column No. 8914 (wedged) carried 424 000 pounds; Column No. 8915 (not wedged) carried 368 000 pounds, column No. 8916 (wedged) carried 376 000 pounds; column No. 8920 (not wedged) carried 365 000 pounds, column No. 8921 (wedged) carried 345 000 pounds. No other comparisons on this basis are possible. In all cases the load-deformation curves

show that the columns with their ends wedged are somewhat stiffer in the later stages of the test than are the ones not wedged. It is believed that the fixing of the upper end of the column is distinctly desirable as securing a more sure and equal degree of fixidity at the two ends of the column.

17 – <u>Basis of Length–Strength Formulas for Columns.</u> It is now generally conceded that the Euler column formulas have no application to steel columns of a slenderness ratio of L/r = 120 or less. For columns of less slenderness there are advocates of three types of formulas – Rankine's (or Schwartz') formula, Johnson's parabolic formula, and the straight-line formula. Tests at Watertown Arsenal and elsewhere on relatively small columns indicate that test results for columns of L/r less than 120 may be expected

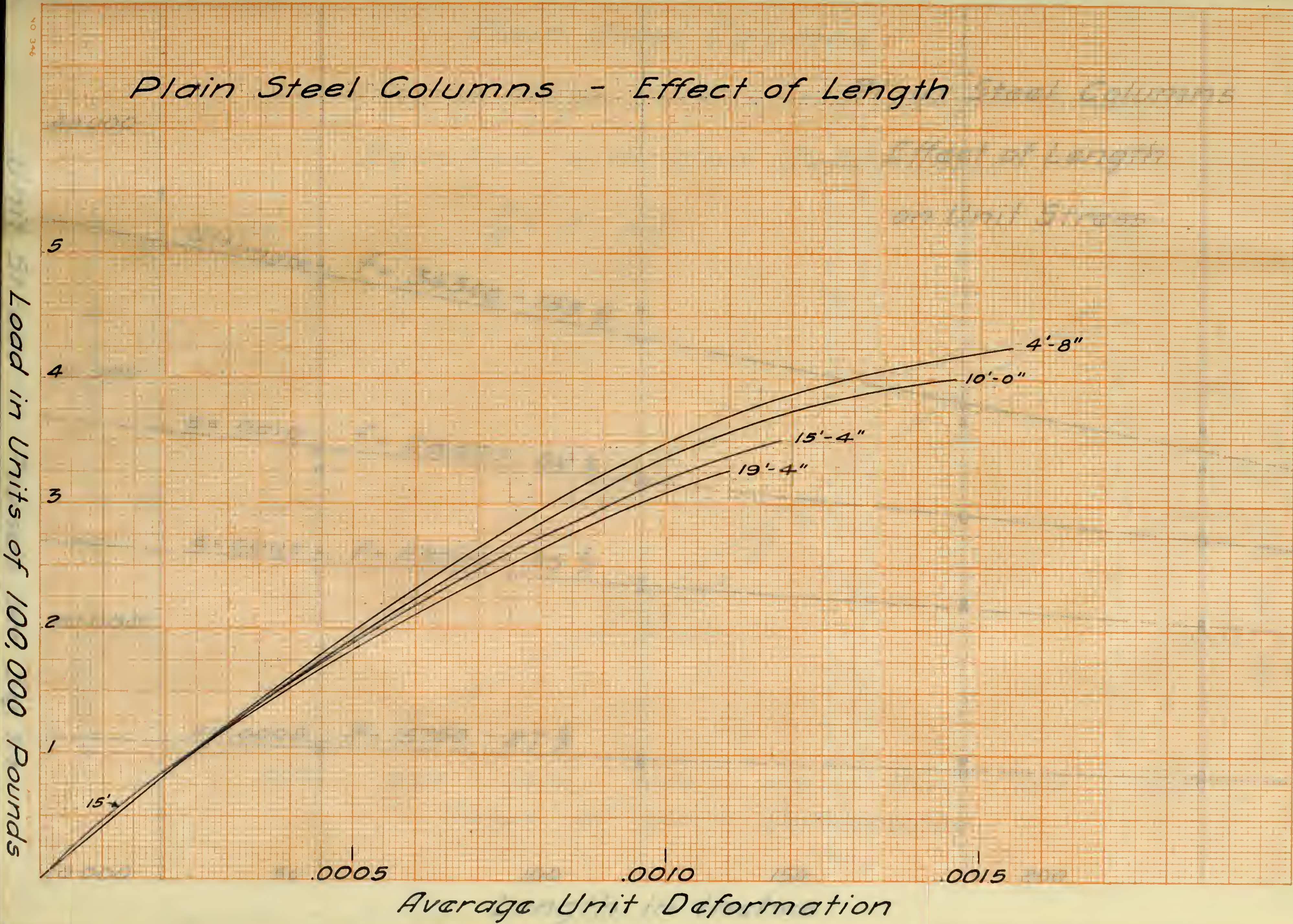


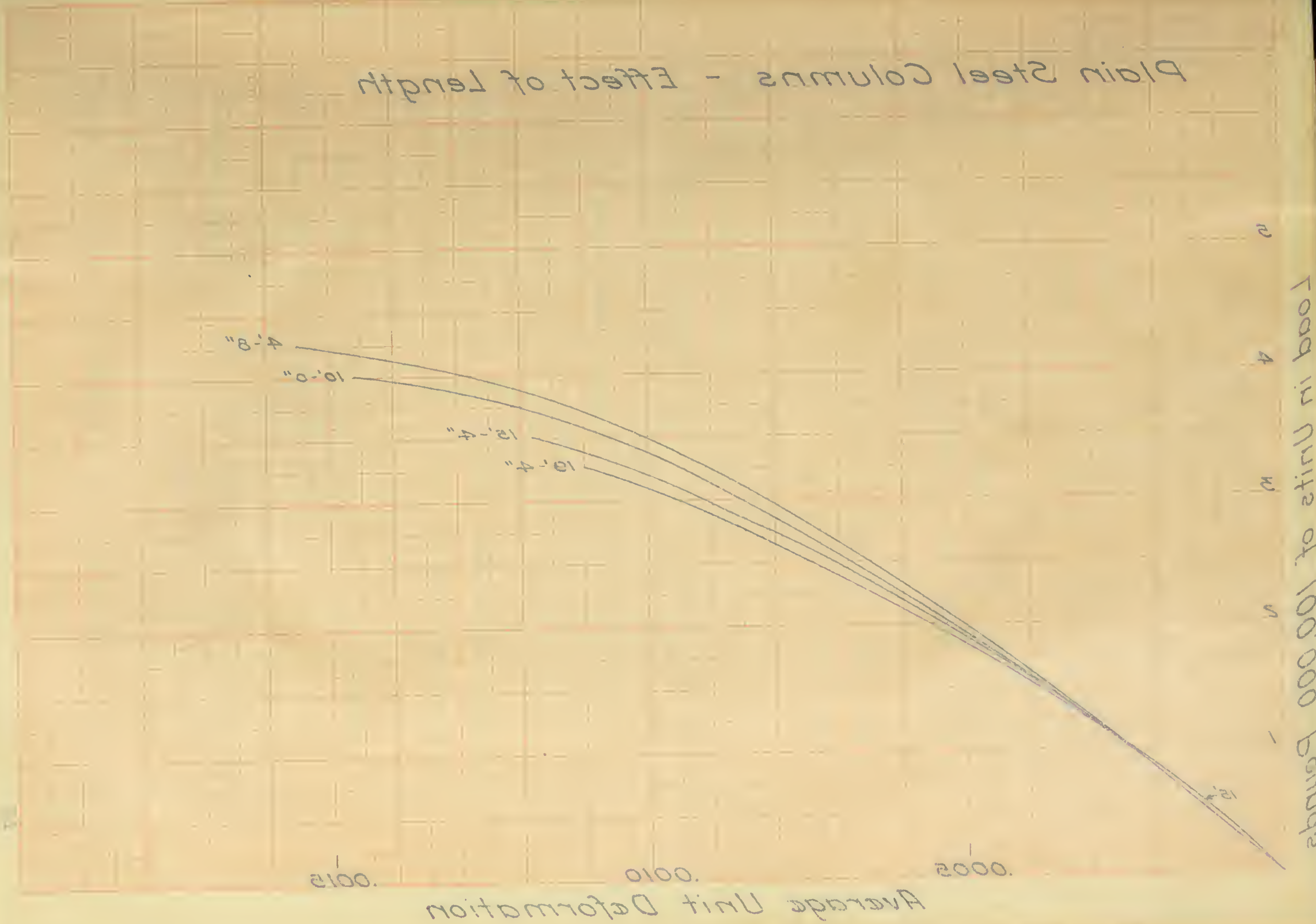
to fall within an area bounded by two parallel straight lines slightly inclined to the L/r axis, and for greater slenderness ratios to agree more closely with the Euler equation. The figure shows the region within which test data may be expected to fall for material with about 40 000 pounds per square inch yield point. In studying the plotted results of the nine plain columns tested, it was found that Johnson's parabolic equation has a

marked curvature ina direction opposite to that indicated in the data of the tests, and hence this formula has not been considered further. In deriving equations to represent the test data it is expressly understood that the equation marks simply the center line of a zone of some width within which the data seems best to fit.

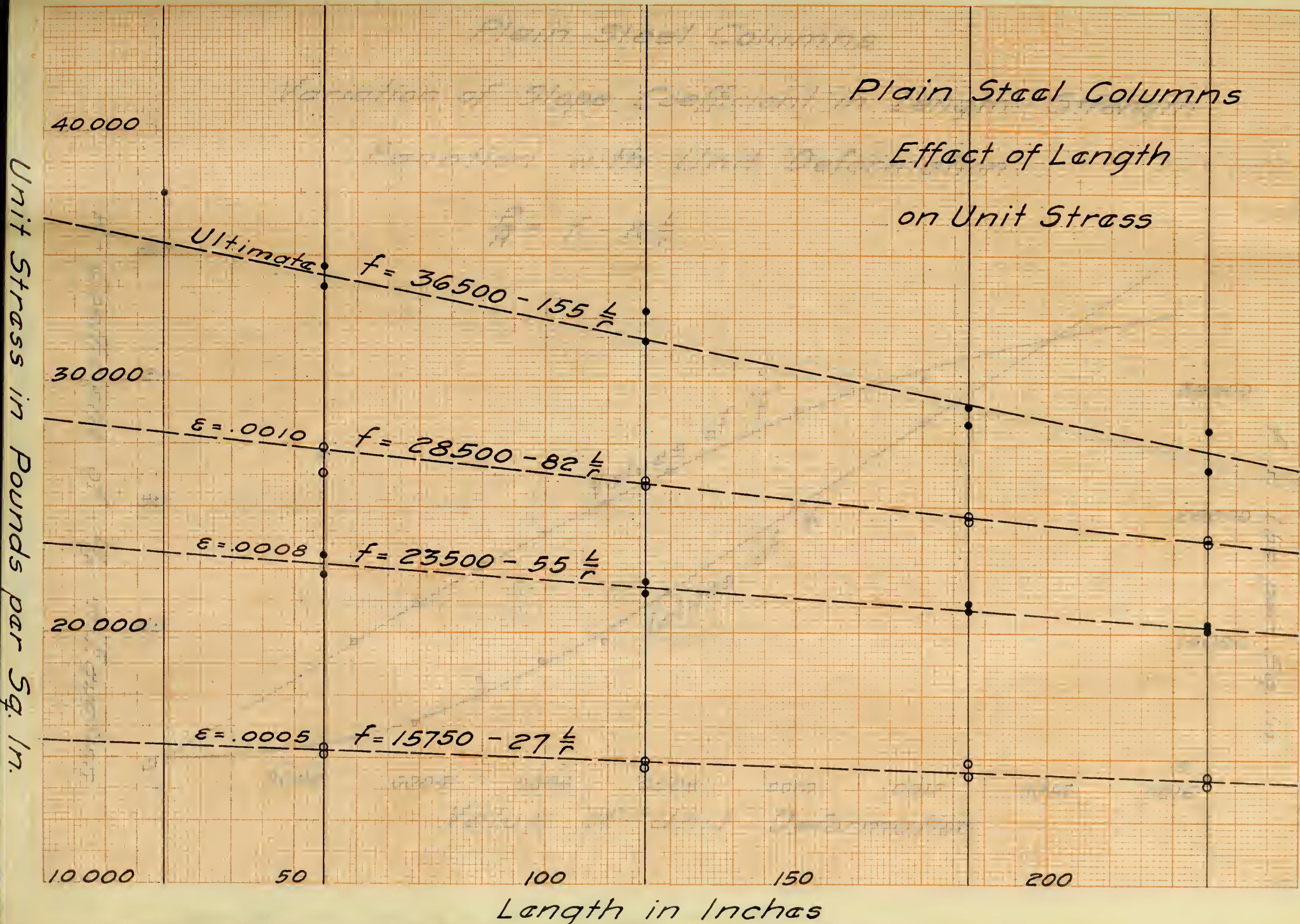
Some notion of the effect of length on strength of column may be obtained from the diagram on page 57 in which the load-deformation curves for the various lengths are plotted together - this is,however, more truly a comparison of stiffness rather than of strength, but the two are very closely associated. To secure data on which to base a length-strength formula nine columns were studied.

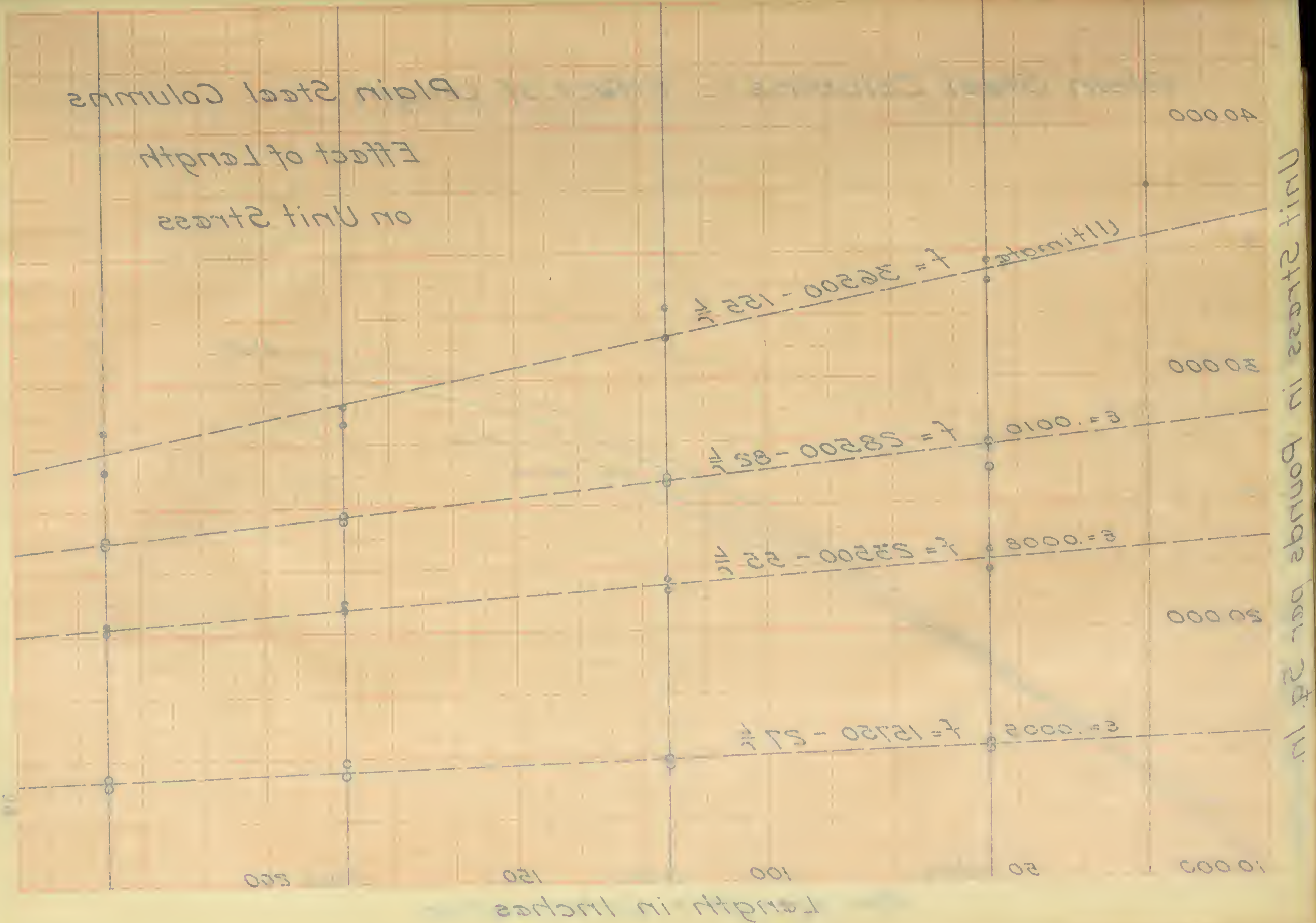
18 - Length-Strength Formulas Fitting Thesis Test Data. In arriving at an expression for the effect of length upon strength it may add to our clearness of perception to trace the development of the effect of length upon strength at various stages of the test. In the diagram on page 58 the results are plotted for four lengths of column and for unit deformations of .0005, .0008, .0010, and for ultimate load. In a similar manner results have been plotted for other unit deformations from .0003 to .0012 and from these data the curves of the diagram on page 59 have been drawn. The lower curve shows very clearly the increasing effect of the slenderness ratio on the strength as the test progresses. If we should produce the tangent back to the horizontal axis the intersection is at .0004 and it is not far from the facts to say that up to a unit deformation of .0004 the slenderness


Plain Steel Columns - Effect of Length
Load in Units of 100,000 Pounds
5
4
3
2
1
4'-8"
10'-0"
15'-4"
19'-4"
15'
.0005
.0010
.0015
Average Unit Deformation



Plain Steel Columns - Effect of Length
4'-8"
10'-0"
15'-4"
19'-4"
15'
.0005
.0010
.0015
Average Unit Deformation
1
2
3
4
5
Load in Units of 100,000 Pounds


Plain Steel Columns
Effect of Length
on Unit Stress
Ultimate
$f = 36500 - 155 \frac{L}{r}$
$\varepsilon = .0010$
$f = 28500 - 82 \frac{L}{r}$
$\varepsilon = .0008$
$f = 23500 - 55 \frac{L}{r}$
$\varepsilon = .0005$
$f = 15750 - 27 \frac{L}{r}$
40.000
30.000
20.000
10.000
50
100
150
200
Length in Inches
Unit Stress in Pounds per Sq. In.

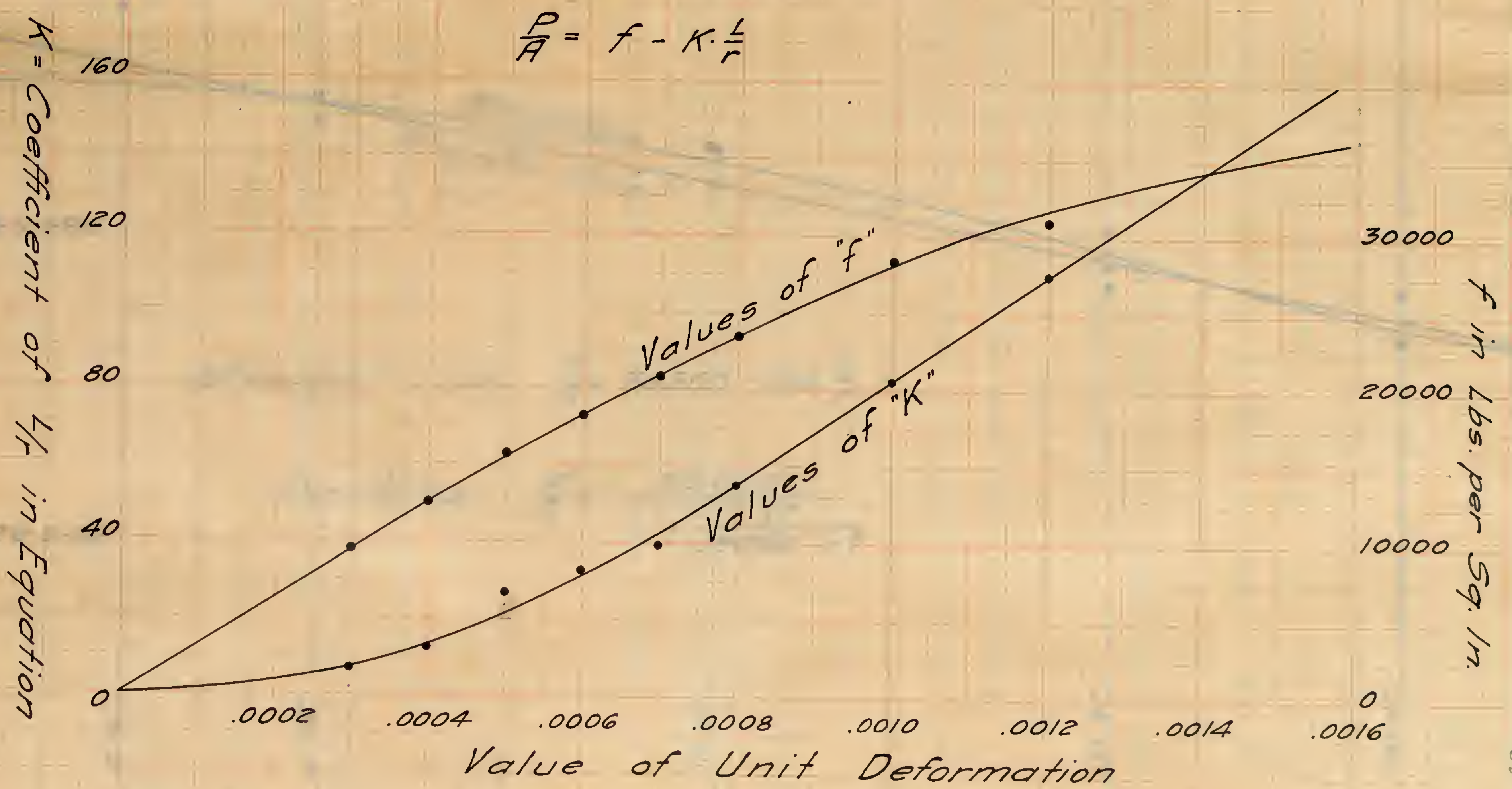


Plain Steel Columns
Effect of Length
on Unit Stress
Ultimate $f = 36500 - 155 \frac{l}{r}$
$\varepsilon = .0010$ $f = 28500 - 82 \frac{l}{r}$
$\varepsilon = .0008$ $f = 23500 - 55 \frac{l}{r}$
$\varepsilon = .0005$ $f = 15750 - 27 \frac{l}{r}$
40 000
30 000
20 000
10 000
50
100
150
200
Length in Inches
Unit Stress in Pounds per Sq. In.

Plain Steel Columns

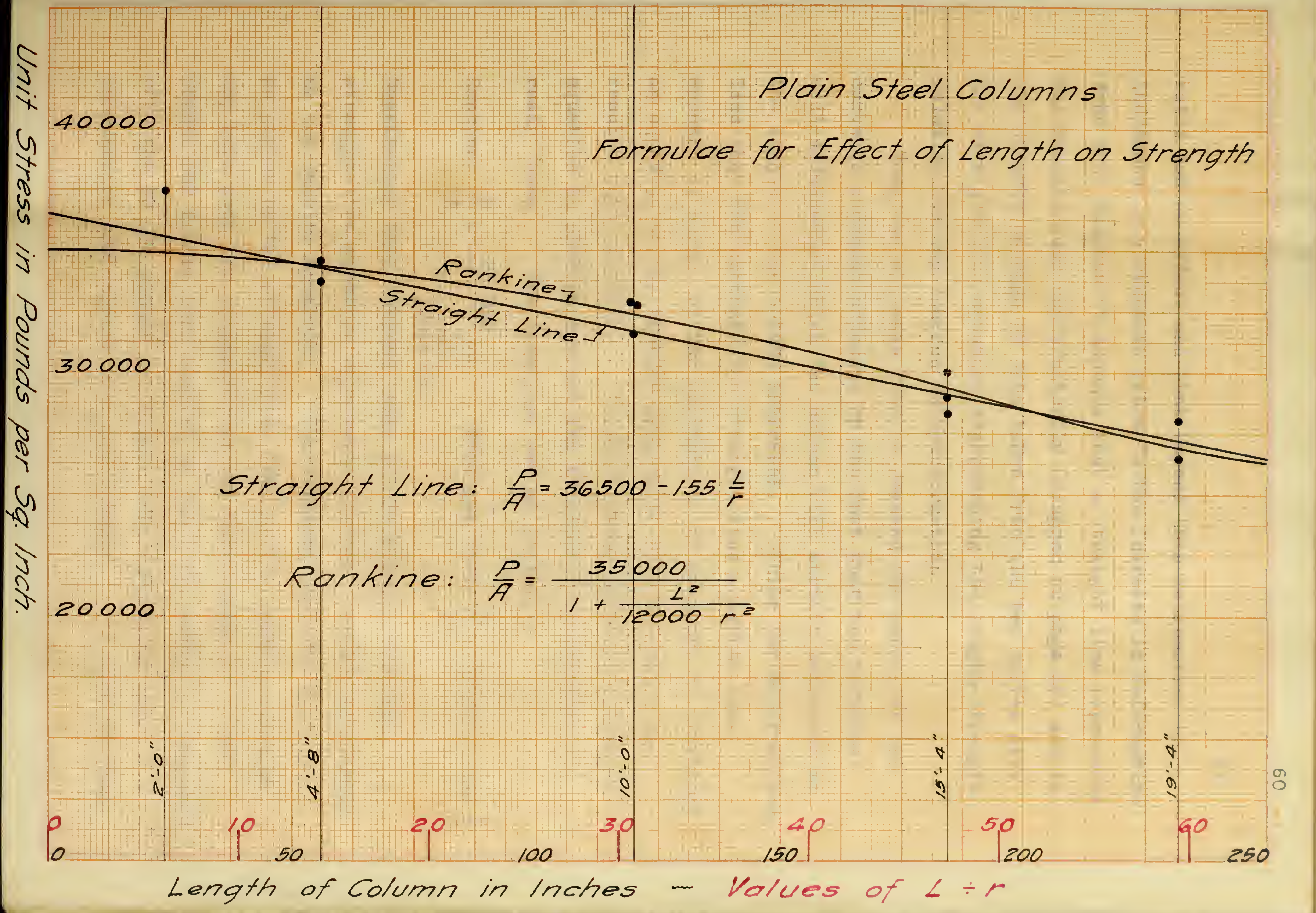
Variation of Slope Coefficient in Length - Strength Equation with Unit Deformation

$$\frac{P}{A} = f - K \cdot \frac{L}{r}$$



No. 346

EUGENE DIETZGEN CO. CHICAGO NEW YORK



Plain Steel Columns
Formulae for Effect of Length on Strength
Rankine
Straight Line
Straight Line: $\frac{P}{A} = 36500 - 155\frac{L}{r}$
Rankine: $\frac{P}{A} = \frac{35000}{1 + \frac{L^2}{12000 r^2}}$
40000
30000
20000
2'-0"
4'-8"
10'-0"
15'-4"
19'-4"
0
10
20
30
40
50
60
0
50
100
150
200
250
Unit Stress in Pounds per Sq. Inch.
Length of Column in Inches — Values of $L \div r$

ratio has no effect and that beyond this deformation it increases in a constant ratio to the increase in deformation. From this diagram it appears that a straight line represents the results very satisfactorily (diagram on page 58) and in the diagram on page 59 data taken from the two curves give the straight-line equation representing the length-strength relation for any desired unit-deformation.

When the ultimate load is reached the nature of the curve is somewhat affected by the fact that the ultimate unit-deformation which the column will sustain decreases as the length of the column increases. While for deformations less than the ultimate the straight-line equation alone appears to apply, at the ultimate it is possible to construct an equation of the Rankine type which will represent the results quite closely. In the diagram on page 60 such an equation is plotted against the straight-line equation already found. The equations are as follows:

Rankine: $\frac{P}{A} = \frac{35\,000}{1 + \frac{L^2}{12000\,r^2}}$ Straight-line: $\frac{P}{A} = 36\,500 - 155\,\frac{L}{r}$

Considering this diagram (page 60) it appears that the straight-line equation represents the test results as closely as the Rankine equation. The ultimate load for $L/r = 0$ is higher by the straight-line than by the Rankine equation and both seem low compared with the unit-stress of 37 500 pounds per square inch carried by the 2 ft. column or compared with the 40 000 pounds per square inch carried by the compression test pieces from the flanges. In view of these

facts, and because of its simple form, the writer believes that the straight-line equation given above may be considered best to represent the effect of length upon ultimate load for the plain steel columns tested in this thesis investigation.

COLUMN NO. 8 9 1 3

FAILURE AT LOWER END

COLUMN NO. 8 9 1 8

SYMMETRICAL FAILURE

B REINFORCED STEEL COLUMNS - EFFECT OF LENGTH

19 - Phenomena of the Tests. The tests of the reinforced steel columns in which the core only was filled with concrete show in general very much the same kind of failure as did the plain steel columns. The columns, as was to be expected, acted more like hooped columns than like columns reinforced with rods only. The capacity for carrying an increasing load under excessive deformations was not present, however. On the other hand the failure was not sudden as in the case of columns reinforced with longitudinal steel rods tied at wide intervals. The columns exhibited much toughness and a very slow failure.

All these columns were tested with the upper bearing block restrained from motion, and in all cases the bending was symmetrical about the center or occurred at some point below the center. The crushing of the concrete, however, was frequently more marked at the top of the column than elsewhere, due to its less density at this place. In the 4 ft. 8 in. columns practically no bending occurred and the column failed in general crushing which was more marked over its upper half. The 10 ft. columns failed finally in bending, No. 8912 bending about the center, while No. 8913 bent sharply near the base (see photograph page 63) The 15 ft. 4 in. and the 19 ft. 4 in. columns failed in bending symmetrically about the center. The concrete crushed on one side of the

column at the center and on the opposite side at the top and bottom. The varying density of the concrete caused by the obstruction to settlement offered by the steel was well shown in the final crushing. In all cases the crushing was more marked at points at or immediately above the tie plates in the steel column than at other points between these plates. Evidently the concrete at these points had been drawn upon during setting to compact the portion below, while the concrete above was restrained from settlement by the steel. As noted above, and as was to be expected, the concrete at the top of the column showed less dense and more susceptible to crushing than that lower down.

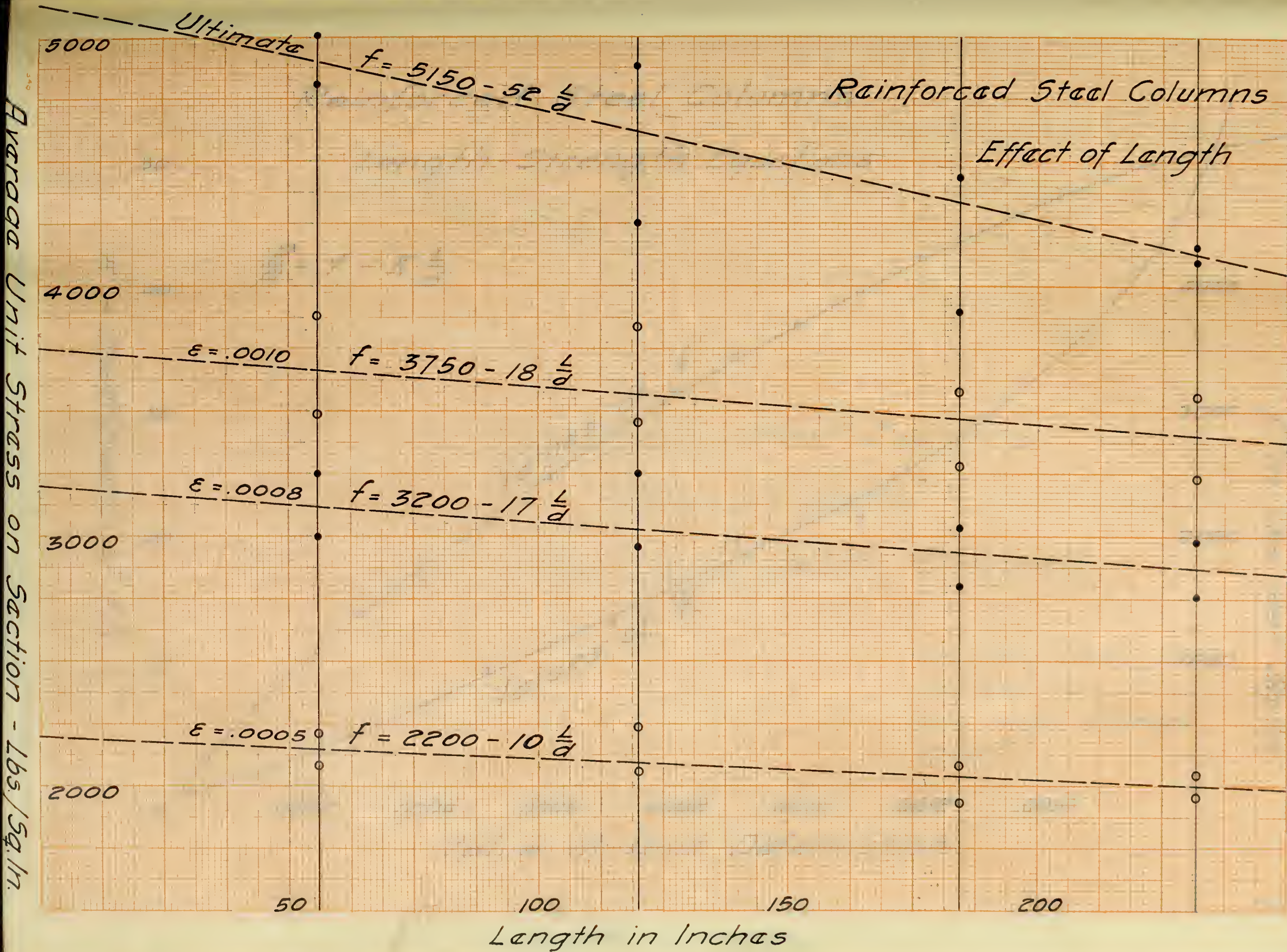
20 - Stress-deformation Relations. It would seem that in the reinforced steel column the conservation of plane section during the test must be quite well maintained and that the error detected in the case of the plain steel columns was not present. This has been assumed to be the case in the interpretation of the data. The load deformation curves are essentially similar to those already viewed for the plain steel columns. In the diagrams (pages 130 to 138) the average load-deformation curve for the two plain steel columns of the same length has been introduced in each case and shows well this marked similarity.

21 - Effect of Test Conditions. In studying the effect of length on the strength of reinforced steel columns it would be best that the length be the only variable. In these columns, however, the strength of the concrete as indicated by the cube and cylinder tests was subject to an extreme

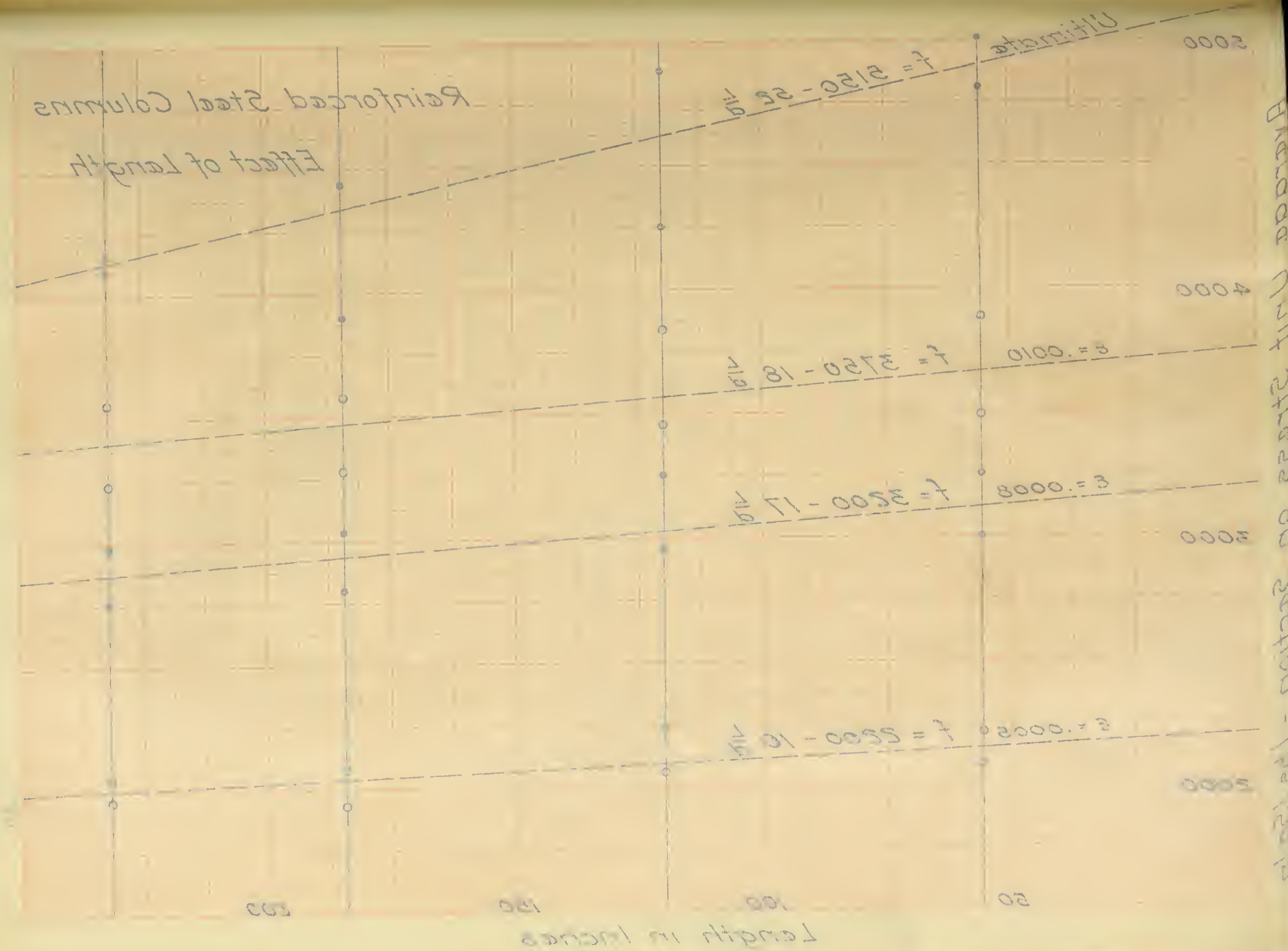
variation of 30 per cent on either side of the mean. It is not feasible to correct for the concrete variation in a satisfactory manner. An attempt has been made to do this in the case of the ultimate loads in paragraph 22, but the basis for such a correction is rather arbitrary at best and it has not been considered worth while to apply it generally.

The thickness of the mortar cushion between the end of the steel column and the top bearing plate may also have exerted an influence on the strength of the columns but it is difficult to arrive at any estimate of the amount. Where the joint was extremely thin, as was generally the case, its effect was undoubtedly negligible, but in one or two cases where this joint was nearly an eighth of an inch thick it may have exerted an appreciable influence on the strength developed by the column.

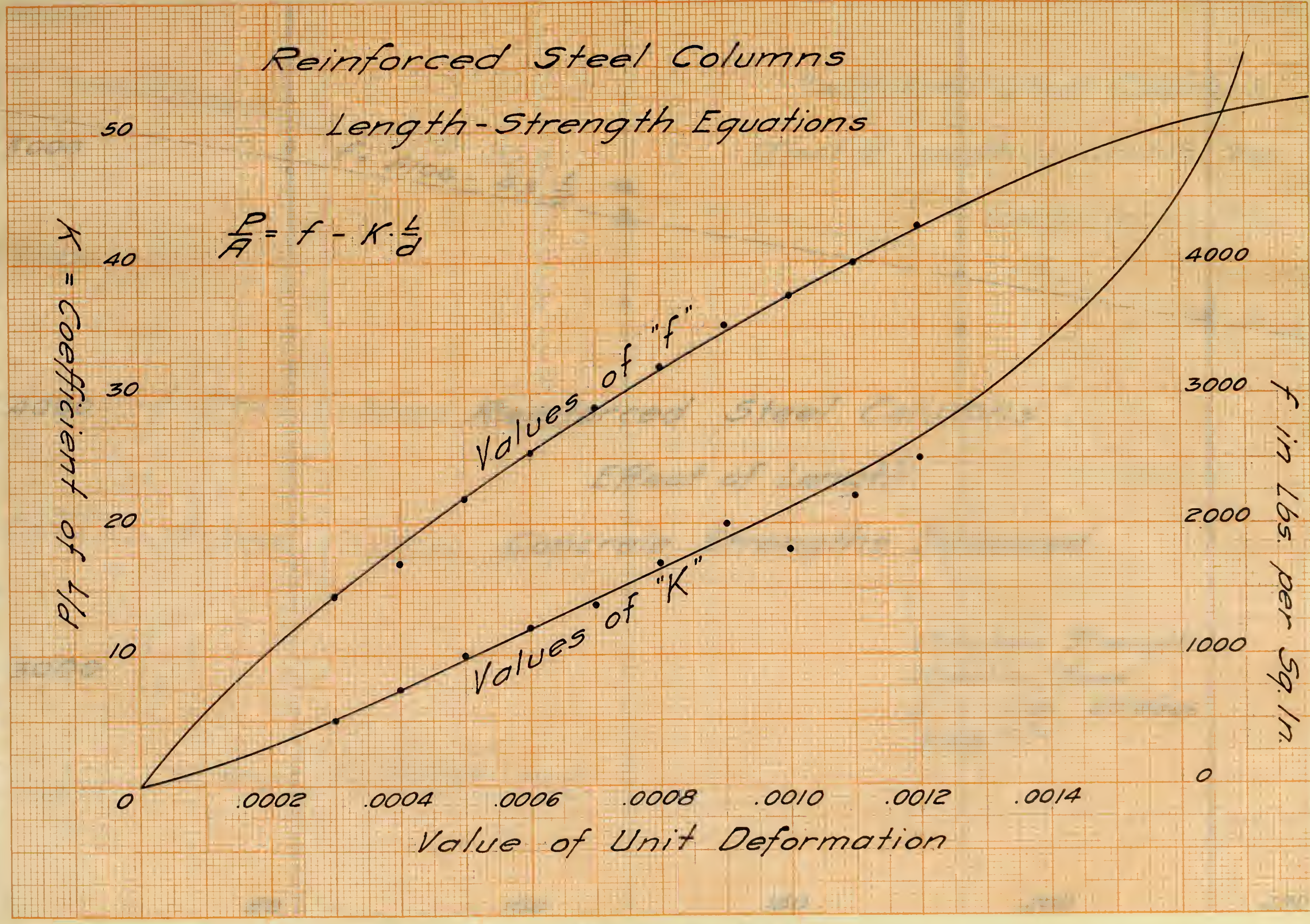
22 - Effect of Length. As in the case of the plain steel columns it may be profitable to study the effect of slenderness (expressed here in terms of L/d) on the strength at various unit-deformations. The diagram on page 68 gives the plotted data for eight columns for unit-deformations of .0005, .0008, and .0010, and also for ultimate load. Owing to the large variation in the concrete strength these points cannot be expected to show as close an agreement as did the plain steel columns. From similar graphical study the equations for unit-deformations from .0003 to .0012 were obtained and the diagram on page 69 gives the curves for the variation of the effect of length (or slenderness) on strength at various stages in the test. From the

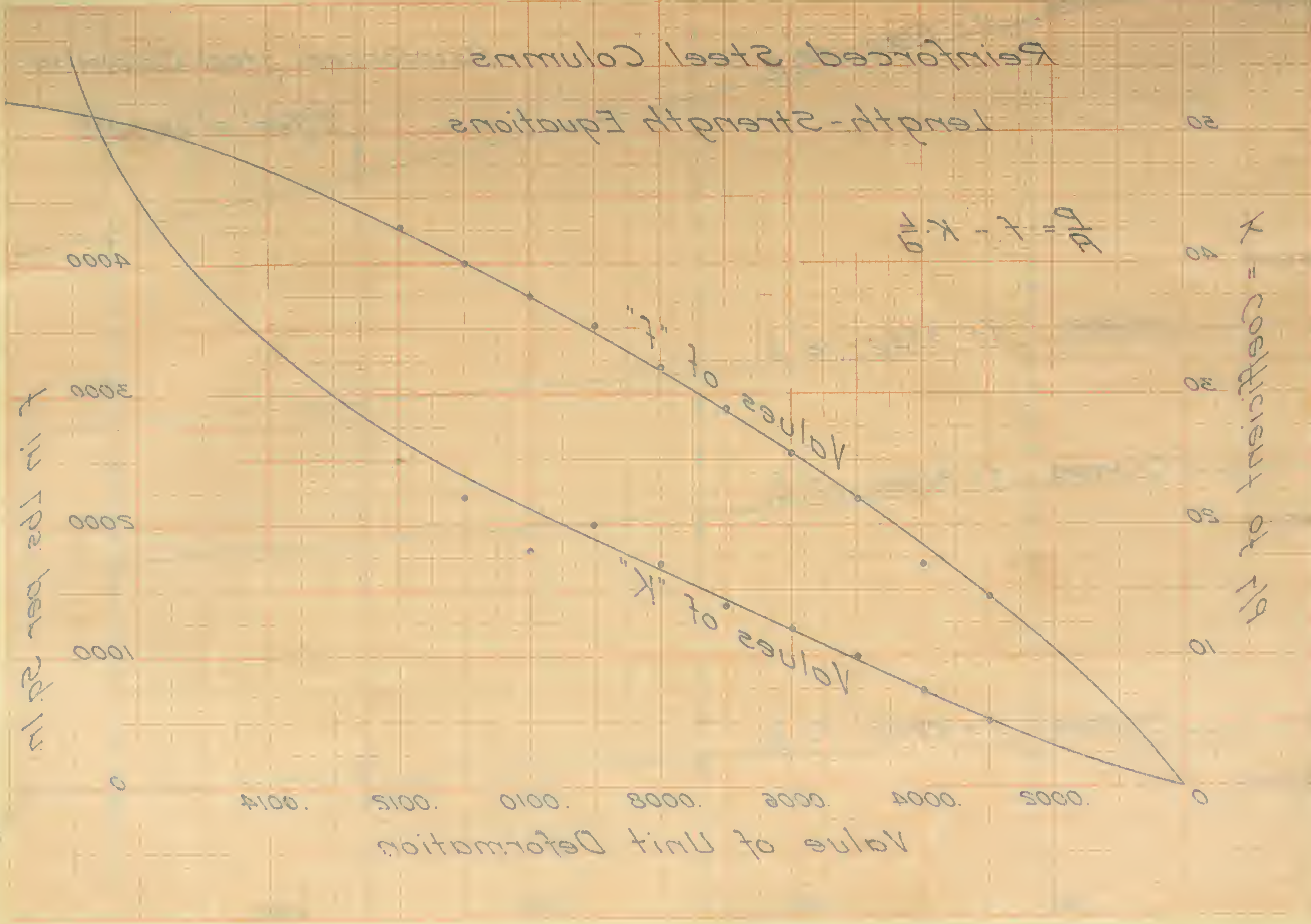


Rainforced Steel Columns
Effect of Length
Ultimate
f = 5150 - 52 L/d
ε = .0010
f = 3750 - 18 L/d
ε = .0008
f = 3200 - 17 L/d
ε = .0005
f = 2200 - 10 L/d
5000
4000
3000
2000
50
100
150
200
Length in Inches
Average Unit Stress on Section - Lbs./Sq.In.

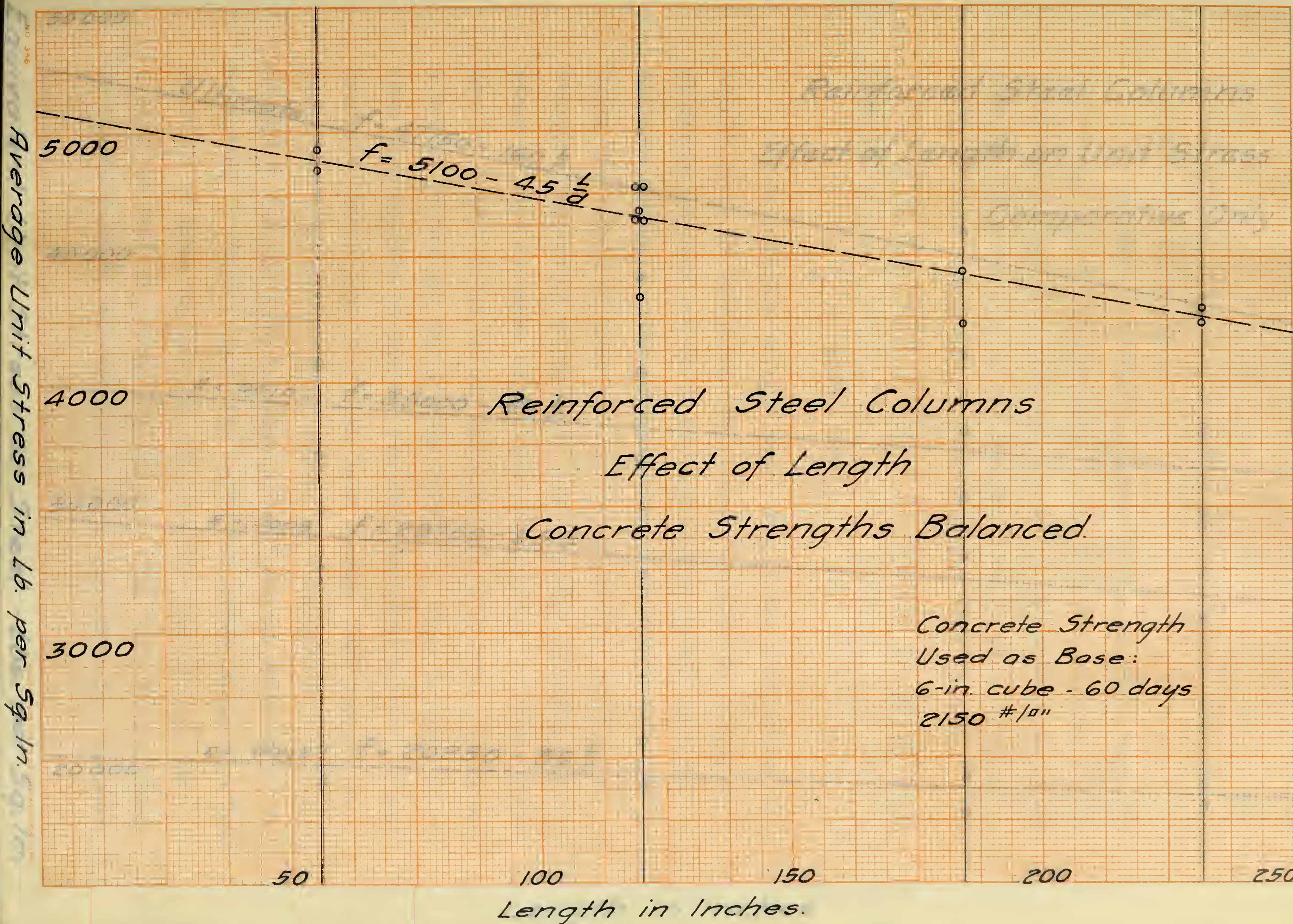


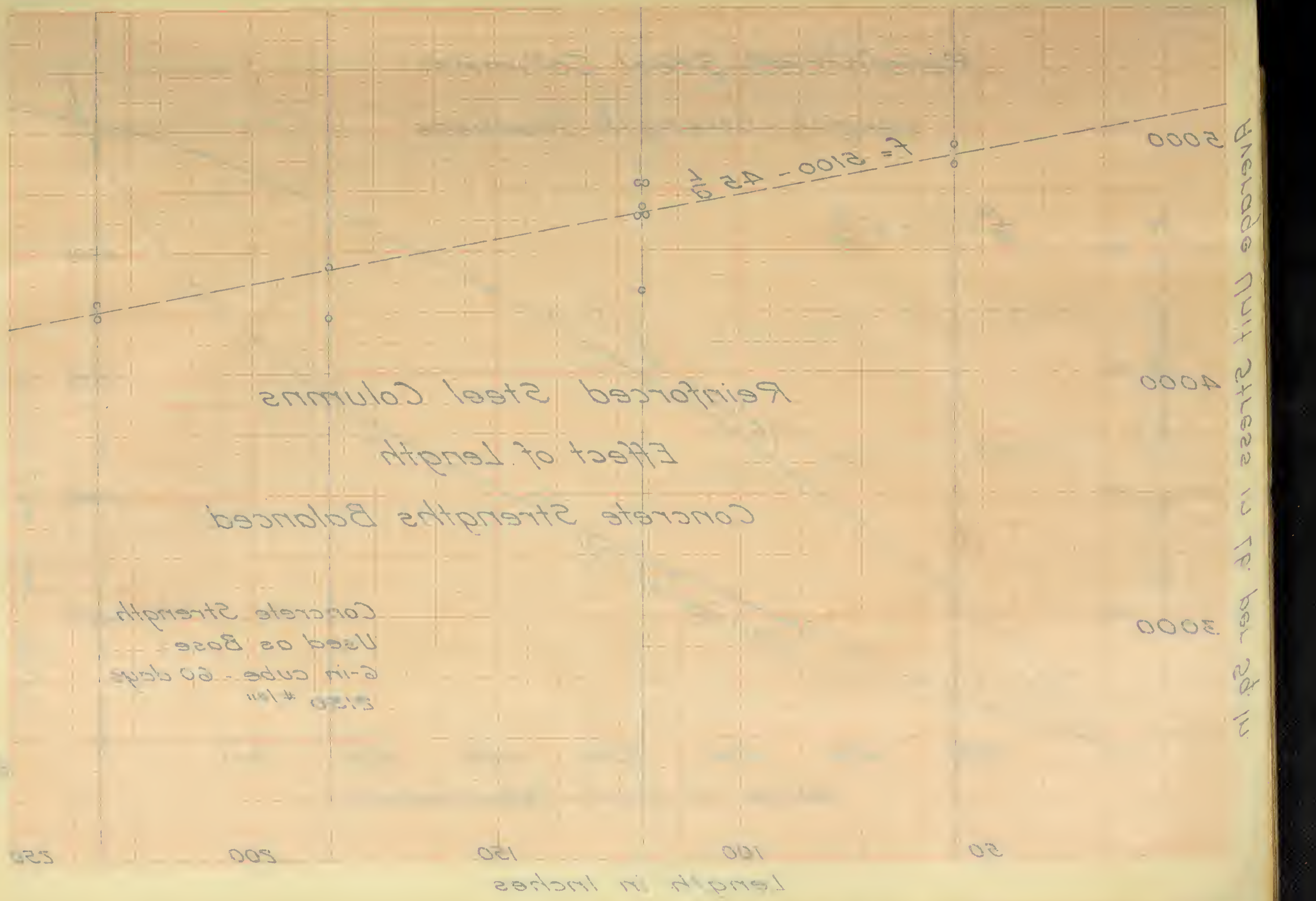
Reinforced Steel Columns
Effect of Length
Ultimate
f = 5150 - 52 l/d
ε = .0010 f = 3750 - 18 l/d
ε = .0008 f = 3200 - 17 l/d
ε = .0005 f = 2200 - 10 l/d
5000
4000
3000
2000
50
100
150
200
Length in Inches
Average Unit Stress on Section - lbs./sq.in.

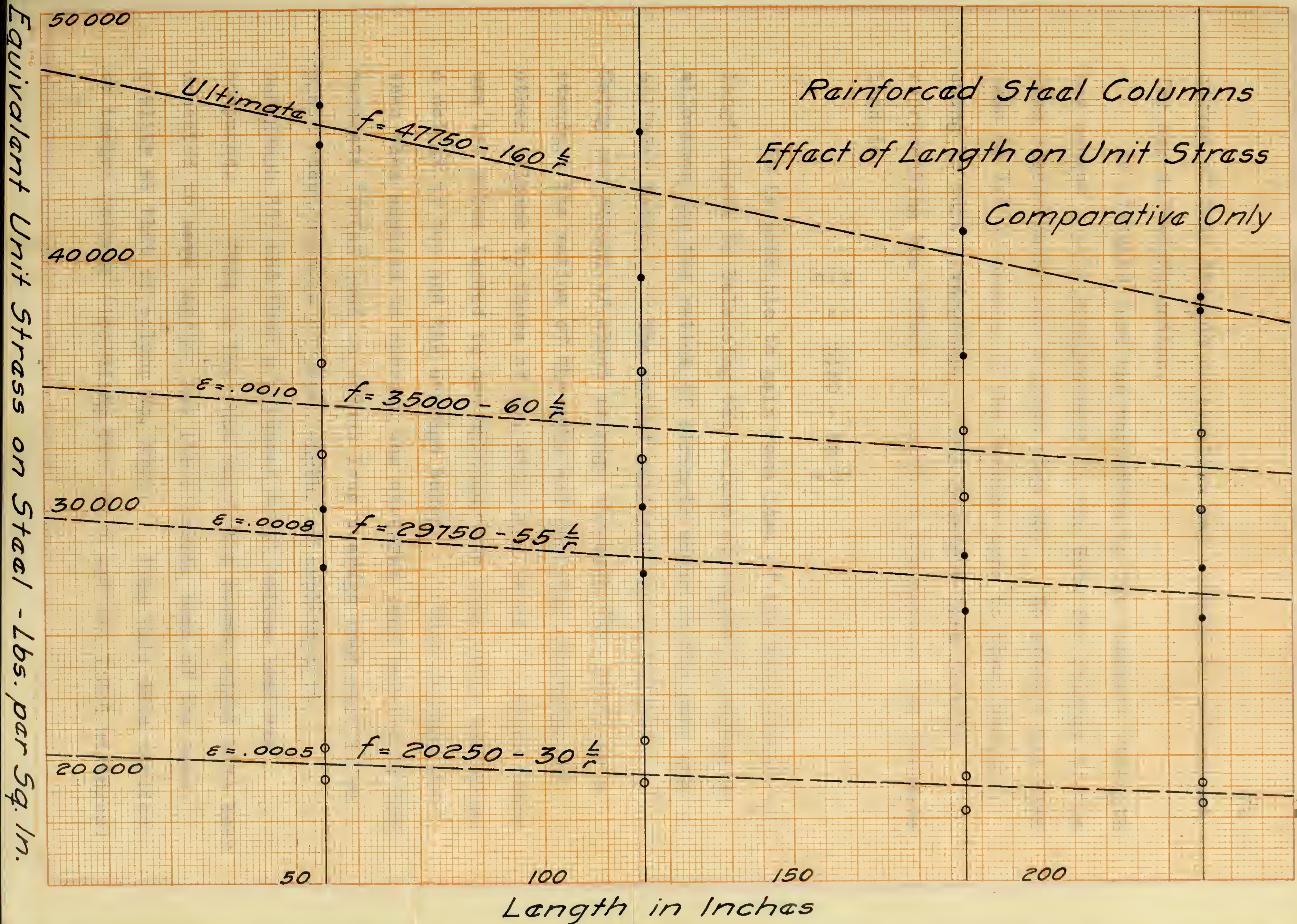

Reinforced Steel Columns
Length-Strength Equations
$\frac{P}{A} = f - K \cdot \frac{L}{d}$
Values of "f"
Values of "K"
K = Coefficient of L/d
f in Lbs. per Sq. In.
Value of Unit Deformation
0
10
20
30
40
50
.0002
.0004
.0006
.0008
.0010
.0012
.0014
1000
2000
3000
4000

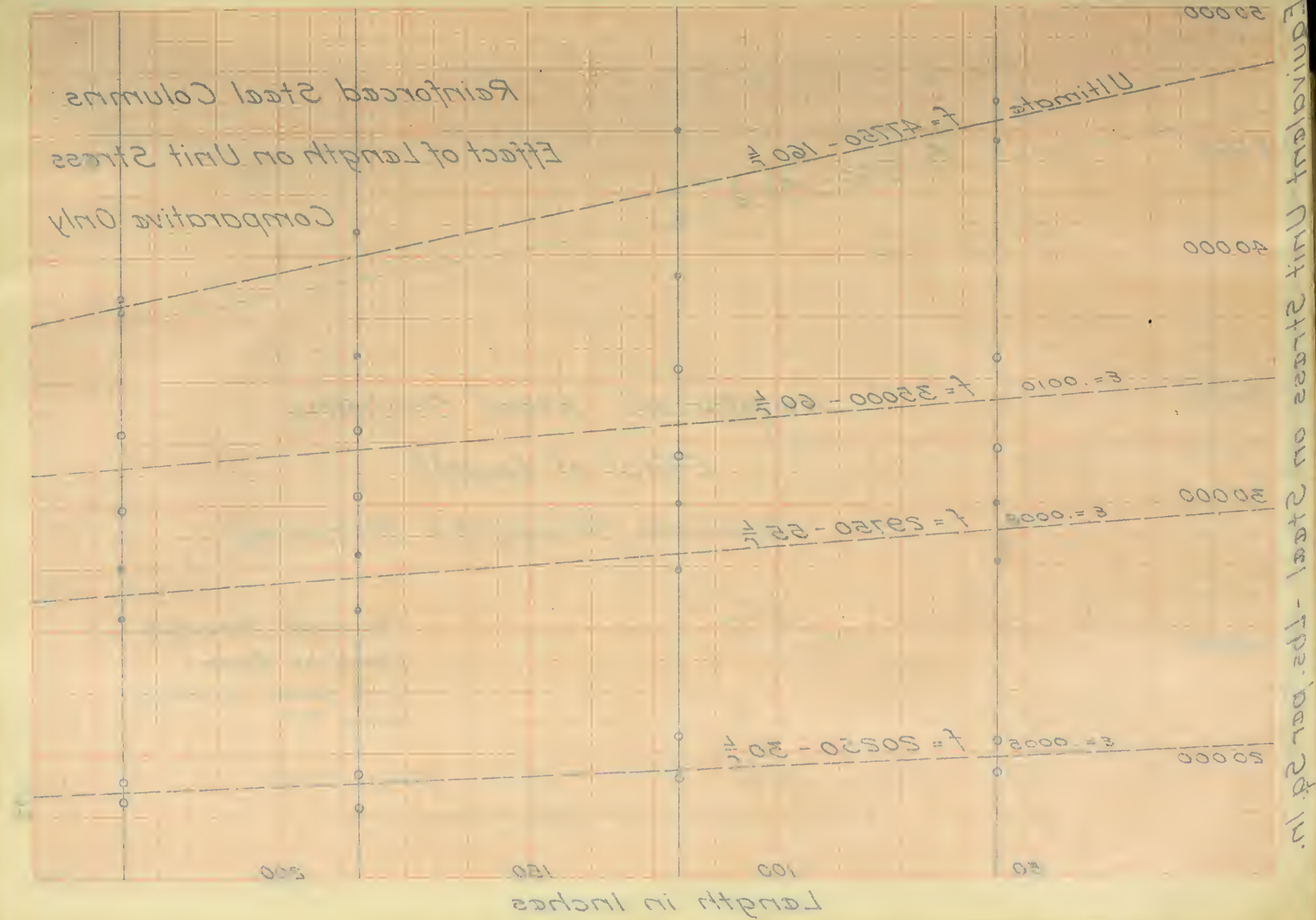


Reinforced Steel Columns
Length-Strength Equations
$\frac{P}{A} = f - K \cdot \frac{l}{d}$
Values of "f"
Values of "K"
f in lbs per sq. in.
K = Coefficient of l/d
Value of Unit Deformation


Reinforced Steel Columns
Effect of Length
Concrete Strengths Balanced
$f = 5100 - 45\frac{L}{d}$
Concrete Strength
Used as Base:
6-in. cube - 60 days
2150 #/□"
5000
4000
3000
50
100
150
200
250
Length in Inches.
Average Unit Stress in Lb. per Sq. In.


Reinforced Steel Columns
Effect of Length
Concrete Strengths Balanced
Concrete Strength
Used as Base
6-in cube - 60 days
2150 #/in"
f = 5100 - 45 L/R
5000
4000
3000
Average Unit Stress in Lb per Sq In
50
100
150
200
250
Length in Inches


Reinforced Steel Columns
Effect of Length on Unit Stress
Comparative Only
Ultimate
f = 47750 - 160 $\frac{L}{r}$
ε = .0010
f = 35000 - 60 $\frac{L}{r}$
ε = .0008
f = 29750 - 55 $\frac{L}{r}$
ε = .0005
f = 20250 - 30 $\frac{L}{r}$
50000
40000
30000
20000
50
100
150
200
Length in Inches
Equivalent Unit Stress on Steel - Lbs. per Sq. In.


Reinforced Steel Columns
Effect of Length on Unit Stress
Comparative Only
Ultimate
$f = 47750 - 160 \frac{l}{r}$
$\varepsilon = .0010$
$f = 35000 - 60 \frac{l}{r}$
$\varepsilon = .0008$
$f = 29750 - 55 \frac{l}{r}$
$\varepsilon = .0005$
$f = 20250 - 30 \frac{l}{r}$
50000
40000
30000
20000
50
100
150
200
Length in Inches
Equivalent Unit Stress on Steel - lbs. per Sq. In.

curves of this diagram an equation may readily be determined for any unit-deformation.

At ultimate load the variation in the concrete strength has caused a wide disagreement in the data for columns of the same slenderness (see diagram page 68) No attempt has been made to fit a formula of the Rankine type to these results owing to this divergency. The straight-line formula representing the effect of length on strength at the ultimate load is

$$\frac{P}{A} = 5150 - 52\frac{L}{d}.$$

It is possible to gain some idea of the true effect of length alone by balancing the column strengths by making an allowance for the ratios of strength shown in the cube and cylinder tests. The method employed was as follows: Using one column of about average strength (No. 8918) as a standard the ratios of the cube and cylinder strengths of other columns to those of No. 8918 were found. Since there are two cubes tested to one cylinder the cube ratio was given a weight of two and the average taken. This average was then then applied to correct the observed load carried by the concrete (total load on column less average load carried on plain steel columns of same length, as explained in paragraph 25) and thus a balanced total column load was computed. This is the load which the column might have been expected to ~~have~~ carry had its concrete been of the same quality as that of column No. 8918. From this data applied to twelve columns (including two 1:1:2 and two 1:3:6 mixtures)

the balanced ultimate unit-loads were plotted against the lengths giving the diagram on page 70. The points lie very much nearer together showing that the method accomplishes at least partially its desired result. From these balanced loads the equation expressing the effect of length on strength becomes

$$\frac{P}{A} = 5100 - 45\frac{L}{d}$$

which may be considered a fair basis for columns of the size and type tested and reinforced with concrete having a cube strength at the time of test of 2150 pounds per square inch (6 in. cube).

23 - Comparison of Plain and Reinforced Steel Columns - Effect of Length. In order to secure a comparison of the relative effect of slenderness in columns both with and without the concrete core, the diagram on page 71 was drawn. In this diagram the concrete was not counted as taking stress but all the load was considered as carried by the plain steel column. Using values of L/r for this plain steel column and the equivalent unit-stress on the steel a diagram may be plotted similar to that on page 58 (for plain steel columns) and in this manner a conclusion may be reached as to the relative slenderness effects. From this method of comparison it is found that the effect of length on strength is almost identical in amount on the plain and on the reinforced steel columns. Thus at ultimate load the coefficient of L/r is 155 for the plain steel columns and 160 for the reinforced steel columns, while at a unit-deformation of .0008

the two coefficients are identical.

NOTE

As a result of the testing of the reinforced steel columns of core section the writer has been much impressed with the many favorable phenomena exhibited by this type of column when subjected to load. It would seem that this column offers the desired toughness and slow failure for a first class structural member, and that it does not tempt the designer to try to figure on a strength (due to spiral) only attained under excessive deformations with attendant danger to other portions of the structure. In this rspect it is distinctly superior to the reinforced concrete column either with or without spiral.

C REINFORCED STEEL COLUMNS - VALUE OF REINFORCEMENT

24 - Test Phenomena and Stress-deformation Relations for Various Mixtures. The test phenomena observed with test columns reinforced with different mixtures of concrete were not different in any striking particular. In fact, all the columns tested showed substantially the same phenomena, a fact accounted for by the presence of sufficient steel to make the concrete effect relatively small. In none of the columns tested did the total load taken by the reinforcement exceed one-half that taken by the steel, and hence the steel governs the behavior of the column under test. The 1:1:2 columns were found to sustain a greater ultimate unit-deformation than the columns of leaner mixtures. The close similarity of the load-deformation diagrams may be noted in the diagrams on pages 131 to 142, for columns Nos. 8907 to 8928. For a statement of the nominal mixture by parts used in the various columns see Table 3 on page 28.

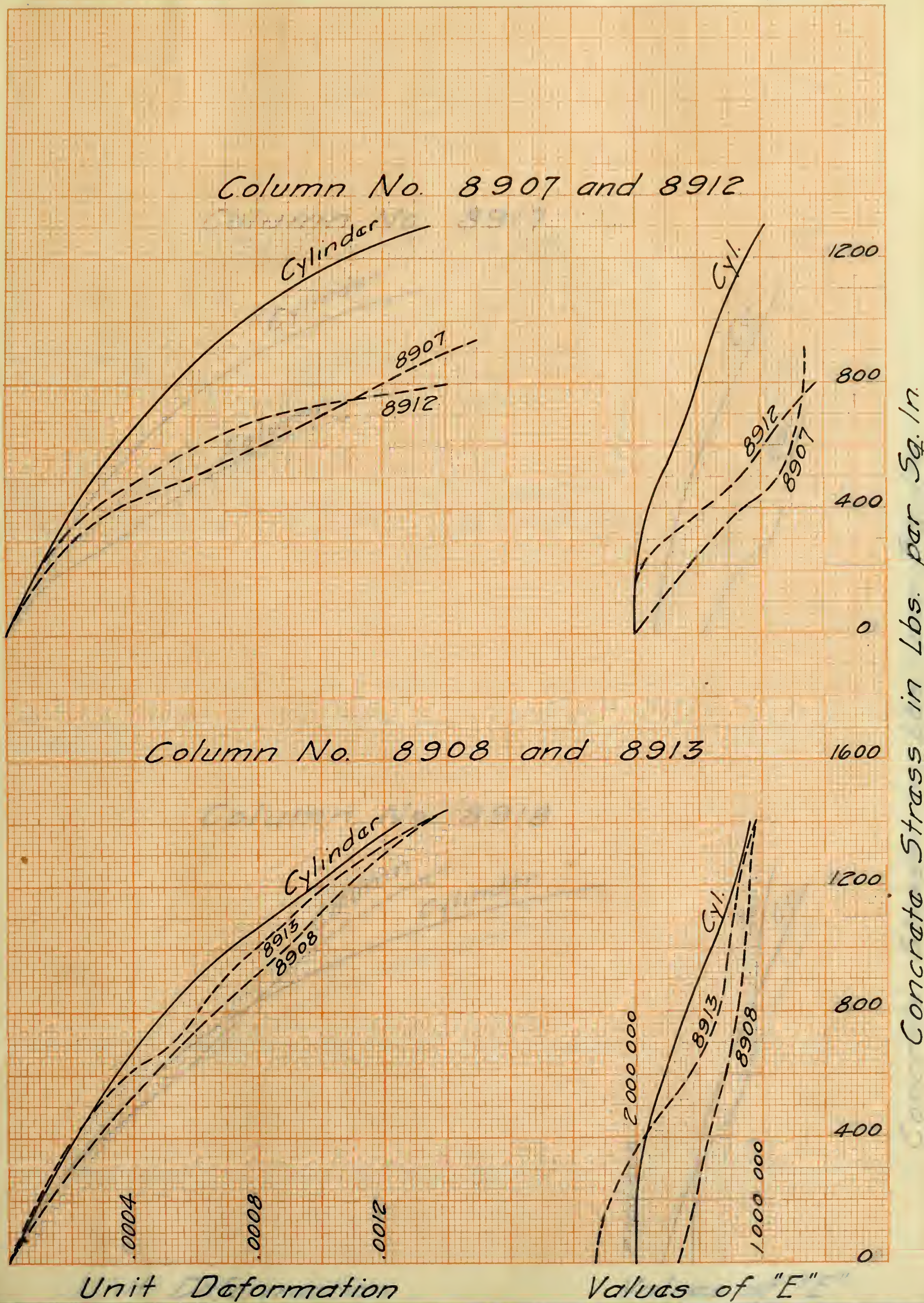
25 - Basis of Curve for Concrete. In studying the strengthening effect of the concrete on the steel column it is necessary to decide upon some basis of division of the total load to be considered as carried by the steel and by the concrete of the columns. In most previous work whether structural steel reinforcement or loose steel rods have been

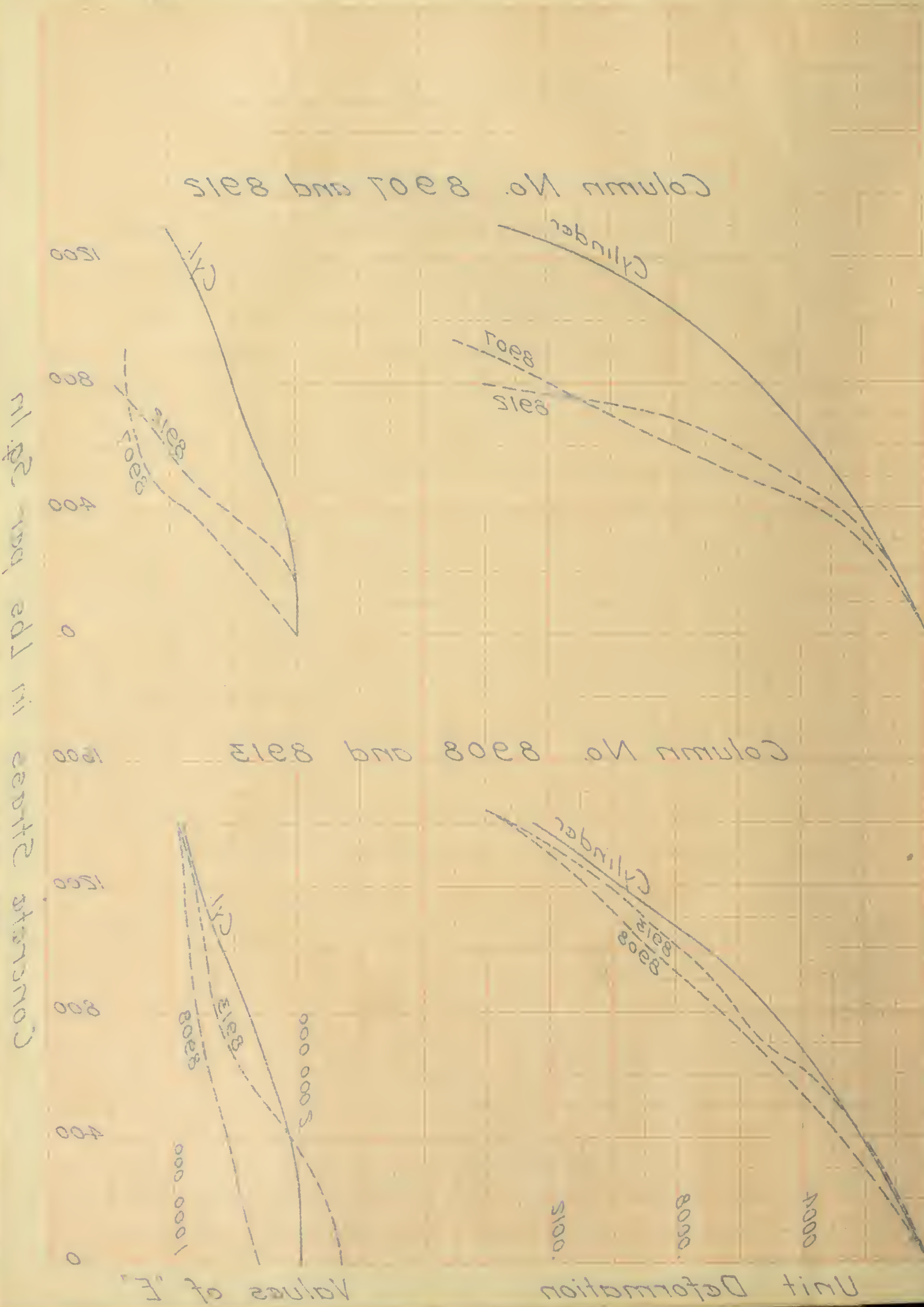
used, it has been customary to attribute to the steel a constant modulus of 30 000 000 pounds per square inch and to determine the load on the steel from the area, the observed deformation, and the modulus. This results in a straight-line diagram for the steel. It assumes a constant modulus for steel in compression within the limits of deformation of the test. It seems to the writer that this assumption is rather unsatisfactory at the later stages of the test even for columns reinforced with rods and with but a low steel percentage. For the structural steel columns it seems evident from previous tests that readjustment in the rivets and tie plates or lacing bars acts to produce a curved stress-deformation line and a varying modulus of elasticity even at comparatively low loads. Whether this variation observed in plain steel columns is overcome in the column reinforced by a core of concrete is a question for which an answer will not be attempted here. It has seemed better for the purpose of this discussion to treat, not of the true distribution of stress between the steel and the concrete of the reinforced steel column, but rather of the increase in load carried by the column at any given unit-deformation, due to the presence of the concrete. To measure this increase there has been plotted on the load-deformation diagrams of all reinforced steel columns the curve for the plain steel column of the same length. For any given unit-deformation we may obtain by simple subtraction the additional load carried by the reinforced column by virtue of the reinforcement. This

item plotted below gives the concrete curve for the various reinforced columns.

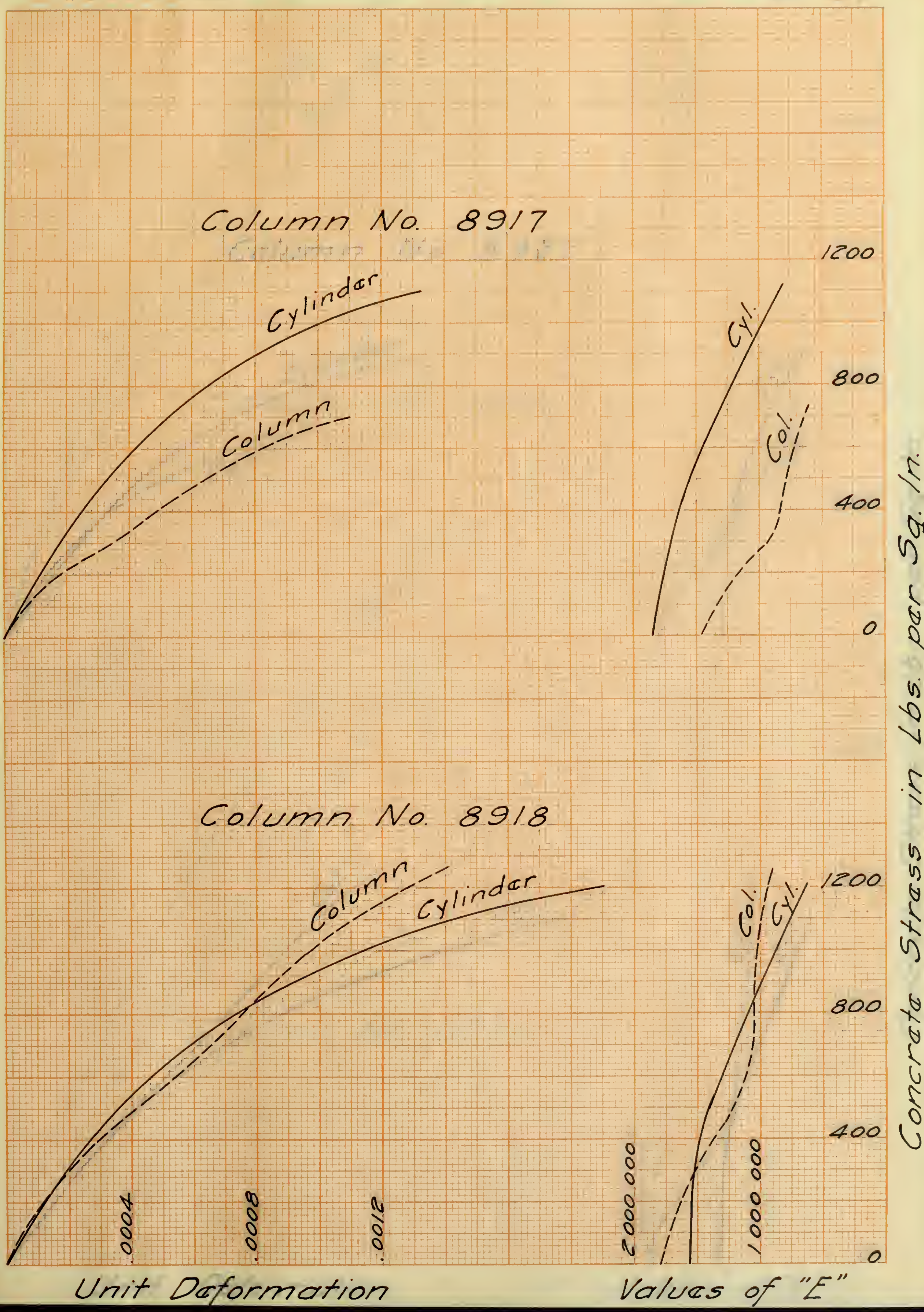
26 – Development and Amount of Concrete Stress. A consideration of these concrete curves enables the unit stress on the concrete to be determined (a comparative, not an absolute, figure) and also the manner of development of this stress. For all mixtures it is found that at first the concrete takes load rapidly, much the same as does the concrete in the cube or the cylinder. The diagrams, pages 78 to 82, offer a comparison of the stress-deformation curves of the same concrete in the column and in the cylinder. It is to be noted that in the early stages the curves are not dissimilar and the moduli of elasticity are nearly the same. As the deformation increases, however, the concrete in the column takes comparatively less and less stress. Disregarding for the moment the question of ultimate strength, it is seen that the same concrete placed in the column is less stiff – has a lower modulus of elasticity – than that in the cylinder. This is the general law but exceptions to it are found. For the 1:1:2 concrete there are no exceptions. For the 1:2:4 mixture six out of eight columns follow the rule. The other two show a reversal at medium loads and in the later stages of the test the concrete in the column shows stiffer than that in the cylinder. For the 1:3:6 mixture both columns follow the latter course.

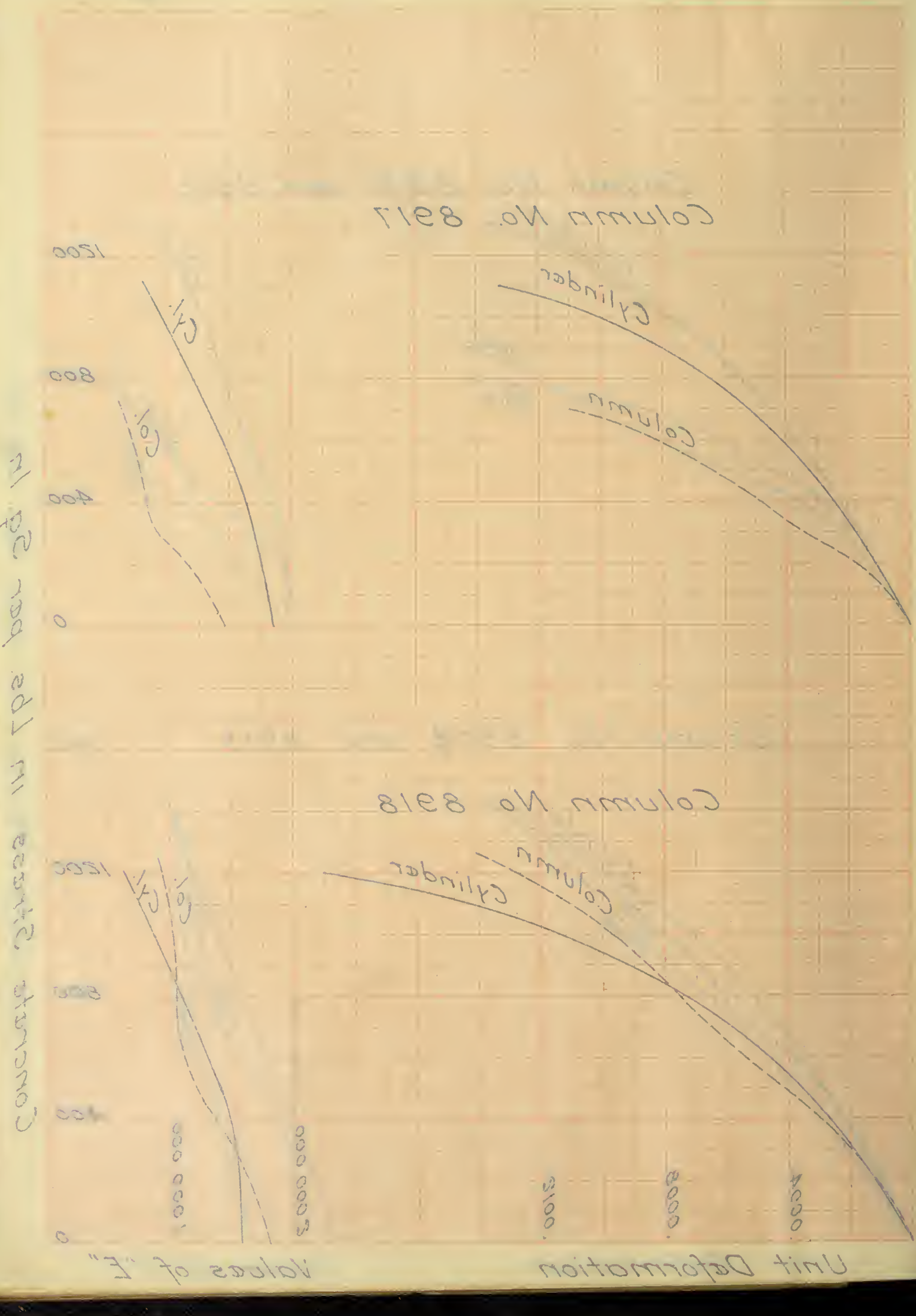
In previous investigations at the Universities of Illinois and Wisconsin it has been observed that the concrete stress, computed as above, shows an increase for a time,


Column No. 8907 and 8912
Cylinder
8907
8912
Cyl.
8912
8907
1200
800
400
0
Column No. 8908 and 8913
Cylinder
8913
8908
Cyl.
8913
8908
2,000,000
1,000,000
1600
1200
800
400
0
.0004
.0008
.0012
Unit Deformation
Values of "E"
Concrete Stress in Lbs. per Sq. In.

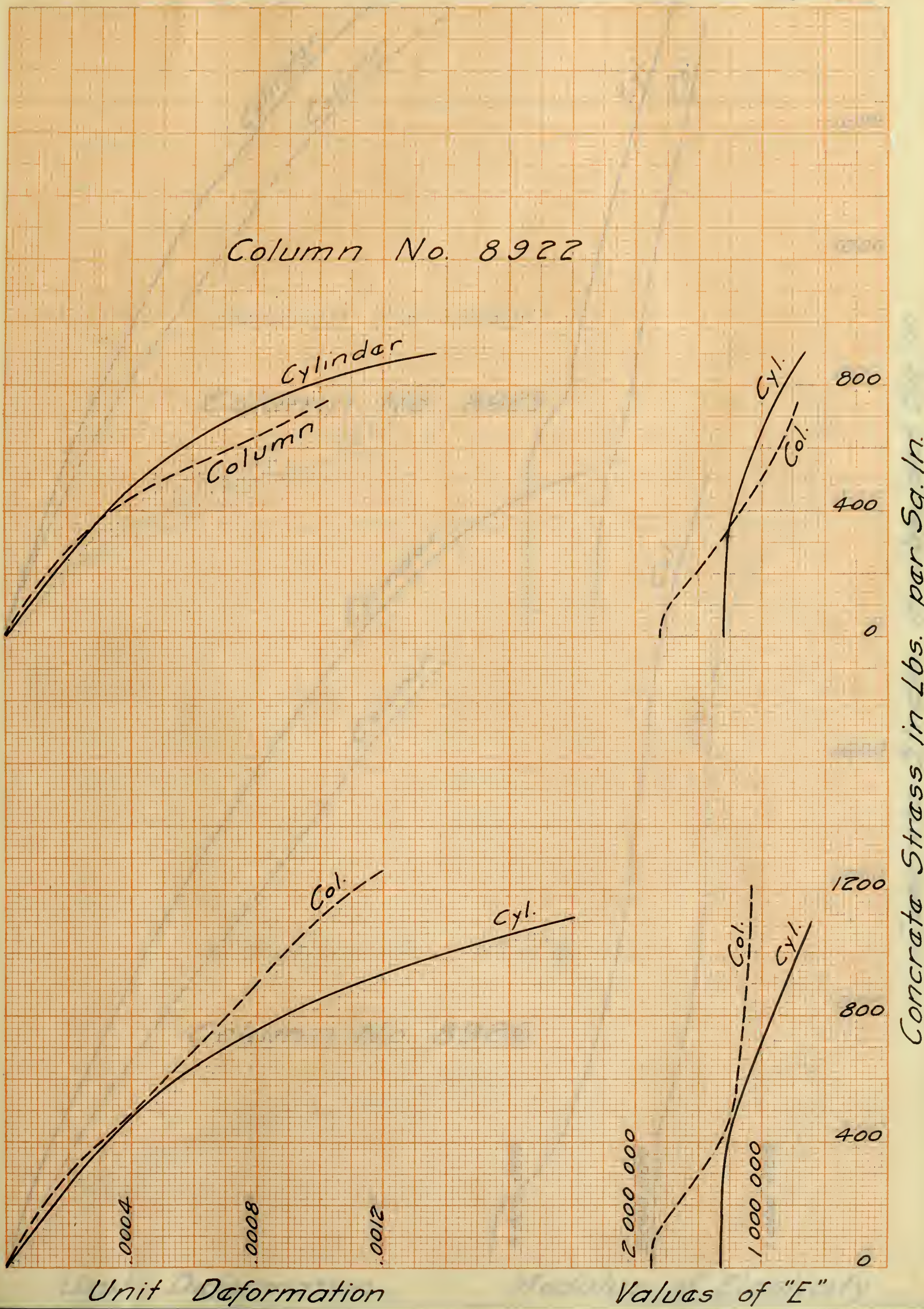


Column No. 8907 and 8912
Cylinder
Cyl.
8907
8912
Column No. 8908 and 8913
Cylinder
Cyl.
8913
8908
Concrete Stress in Lbs. per Sq. In.
0
400
800
1200
1600
1000 000
2 000 000
.0012
.0008
.0004
Values of "E"
Unit Deformation

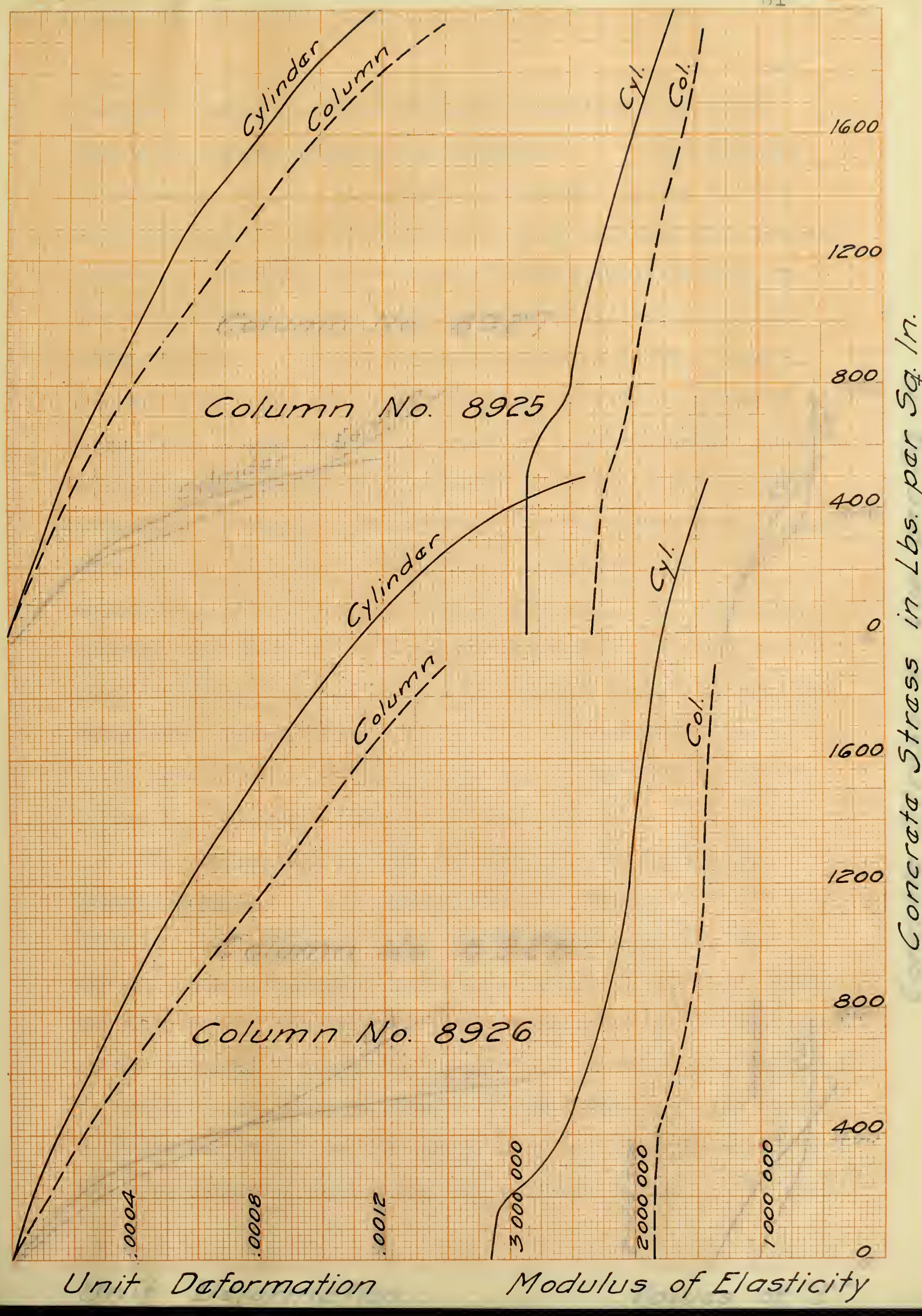

Column No. 8917
Cylinder
Column
Cyl.
Col.
1200
800
400
0
Column No. 8918
Column
Cylinder
Col.
Cyl.
1200
800
400
0
.0004
.0008
.0012
2,000,000
1,000,000
Concrete Stress in Lbs. per Sq. In.
Unit Deformation
Values of "E"

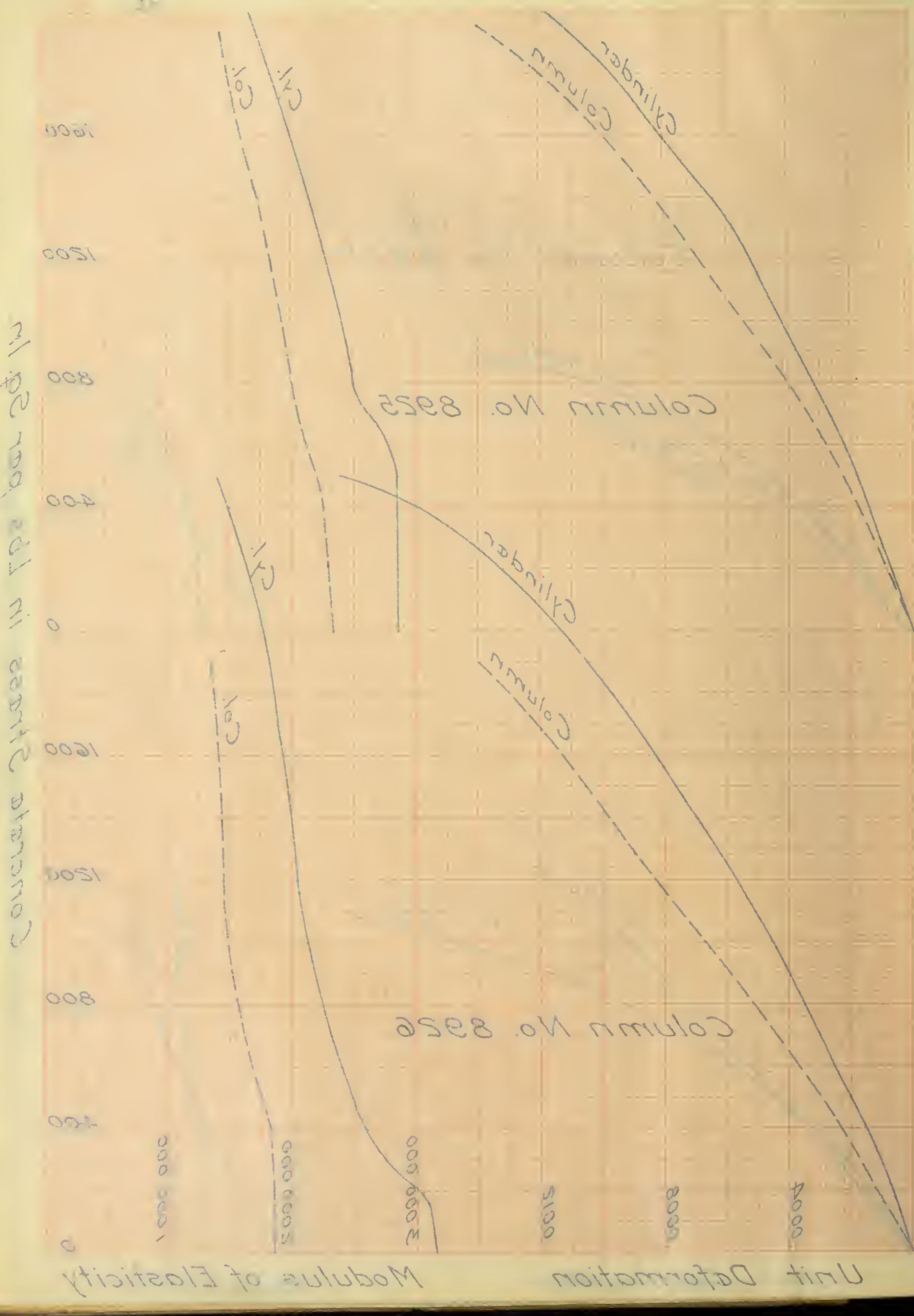


Column No. 8917
Cylinder
Column
Cyl.
Col.
Column No 8918
Cylinder
Column
Cyl.
Col.
Concrete Stress in lbs per sq in
0
400
800
1200
1 000 000
2 000 000
.0004
.0008
.0012
Unit Deformation
Values of "E"

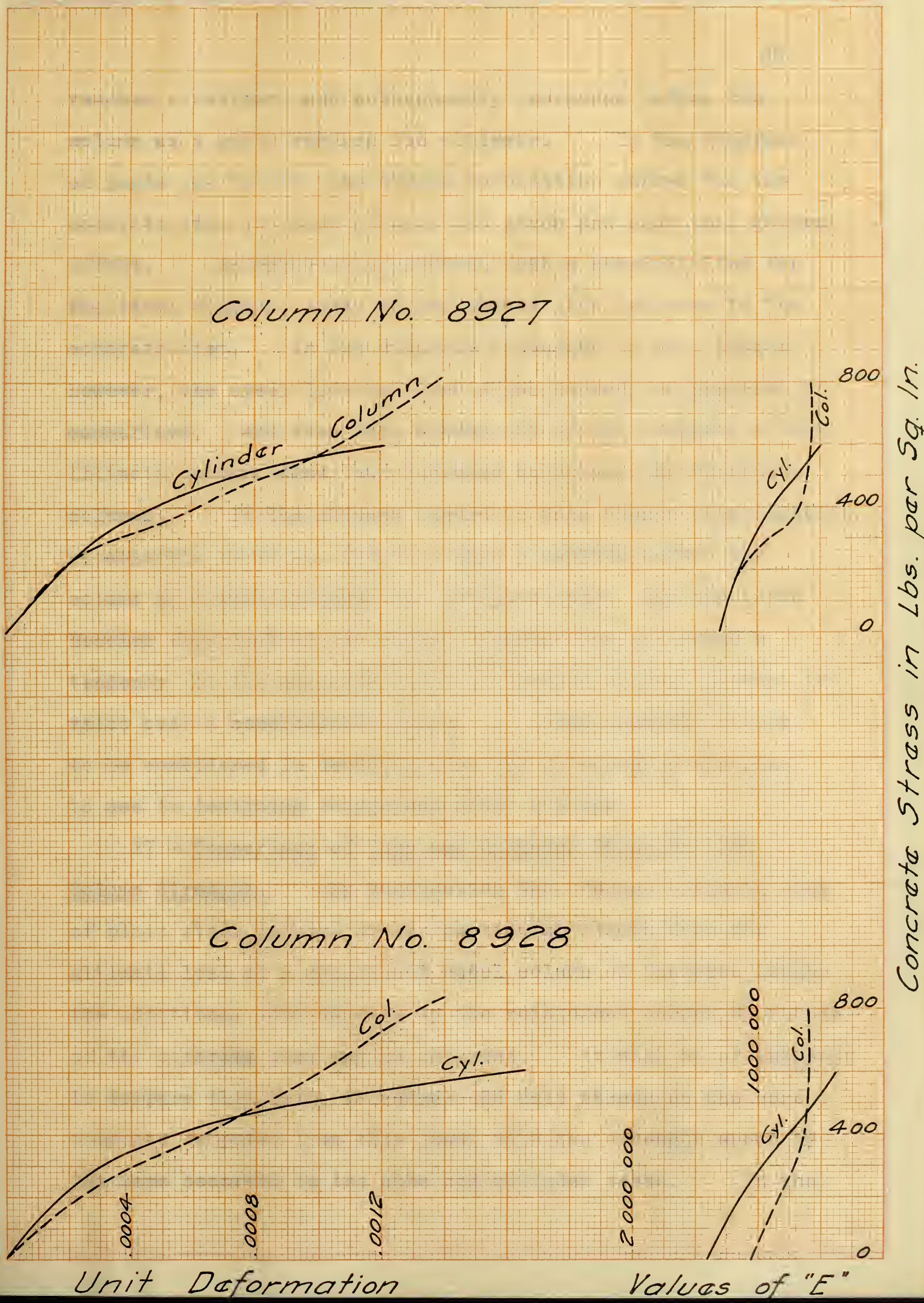

Column No. 8922
Cylinder
Column
Cyl.
Col.
800
400
0
Col.
Cyl.
Cyl.
Col.
1200
800
400
0
.0004
.0008
.0012
2 000 000
1 000 000
Concrete Stress in Lbs. per Sq. In.
Unit Deformation
Values of "E"


Column No 8922
Cylinder
Column
Cyl
Col
Unit Deformation
Values of "E"
Concrete Stress in lbs per Sq In


Cylinder
Column
Cyl.
Col.
Column No. 8925
Cylinder
Column
Cyl.
Col.
Column No. 8926
1600
1200
800
400
0
1600
1200
800
400
0
.0004
.0008
.0012
3 000 000
2 000 000
1 000 000
Unit Deformation
Modulus of Elasticity
Concrete Stress in Lbs. per Sq. In.



Column No. 8925
Column No. 8926
Cylinder
Column
Cyl.
Col.
Concrete Stress in Lbs per Sq. In
Modulus of Elasticity
Unit Deformation
1600
1200
800
400
0
1000 000
2000 000
3000 000
.0012
.0008
.0004

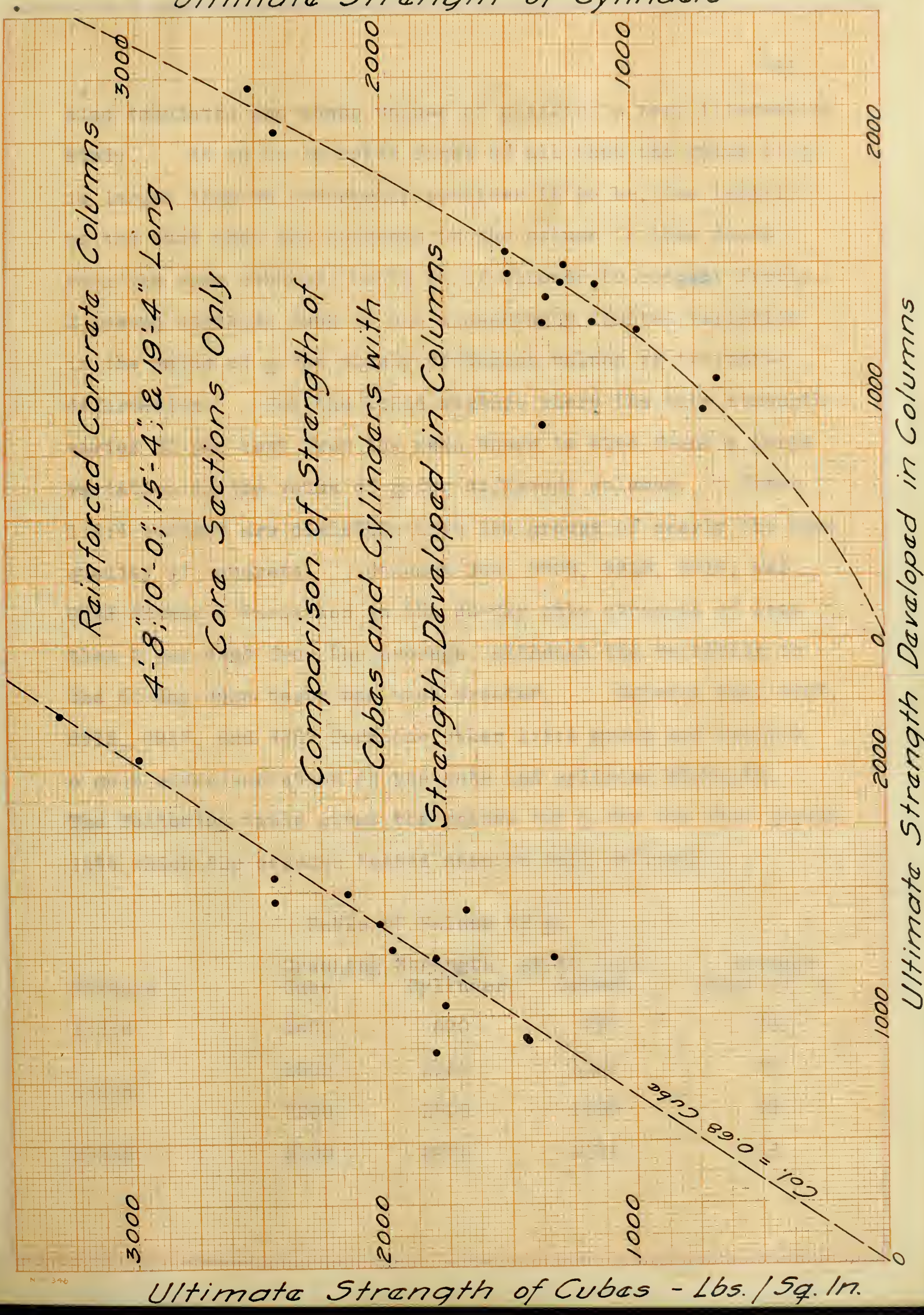

Column No. 8927
Column
Cylinder
Cyl.
Col.
800
400
0
Column No. 8928
Col.
Cyl.
Cyl.
Col.
1000 000
2 000 000
800
400
0
.0004
.0008
.0012
Unit Deformation
Values of "E"
Concrete Stress in Lbs. per Sq. In.

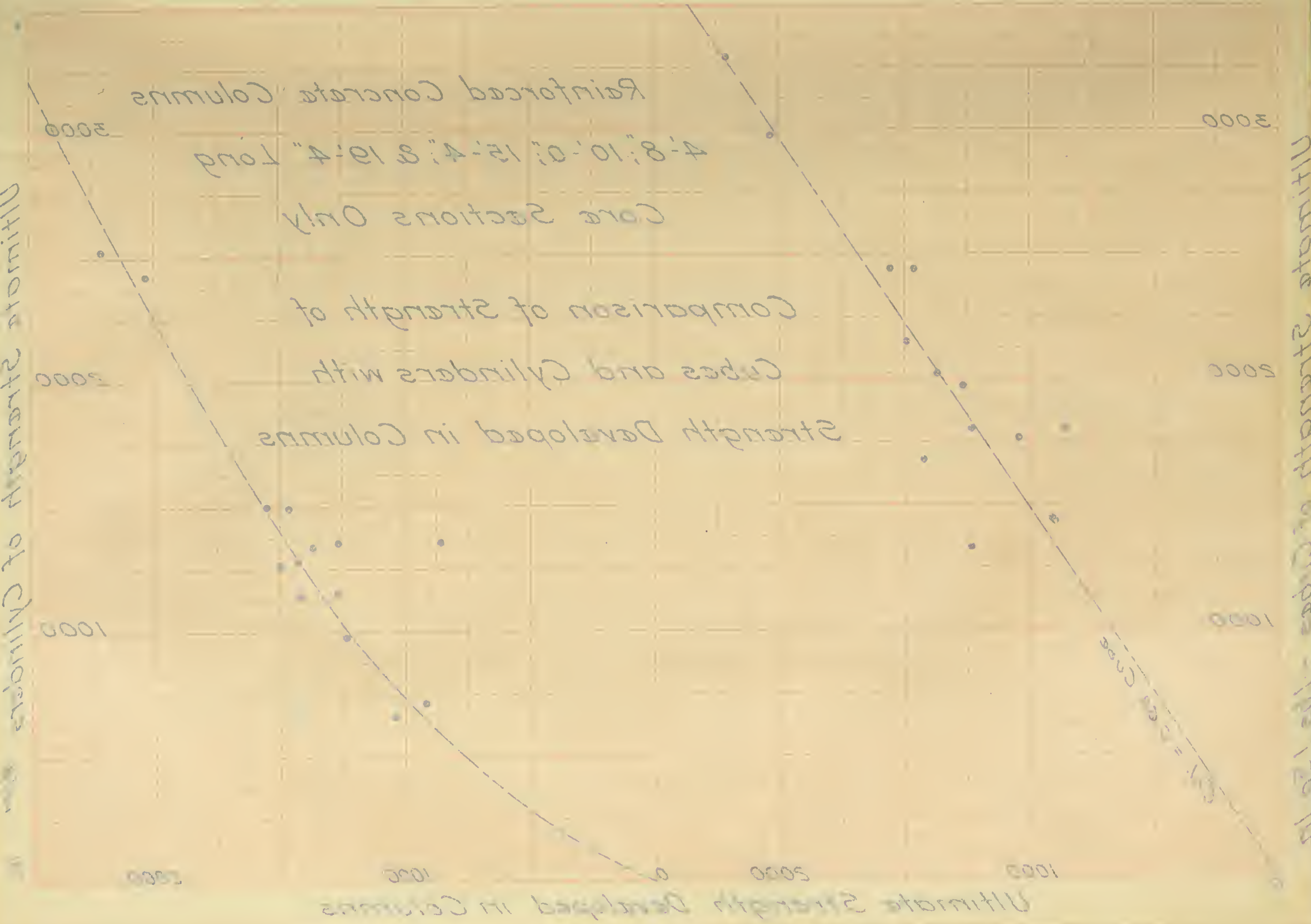
reaches a maximum, and subsequently decreases before the column as a whole reaches its ultimate. In the diagrams on pages 158 to 172, the stress-deformation curves for the concrete item in these columns are given and show this general effect. As originally plotted, with a straight line for the steel diagram, every column showed this decrease in the concrete item. In the diagrams contained in this thesis, however, the steel line has been shown curved for purposes of comparison, and even here eleven out of the fourteen of the Illinois columns show this decrease as do all the Wisconsin columns. In the columns tested in this thesis investigation no concrete curve was found to have a maximum before the column as a whole reached its ultimate load, but the curve becomes very flat at the higher deformations and shows a tendency for the concrete stress to become nearly constant in value over a considerable range. This tendency is one to be considered in deciding upon the permissable stresses to use in designing reinforced steel columns.

27 - <u>Comparison of Cube and Cylinder Strength with Column Strength.</u> By subtracting the average ultimate load of plain steel columns of any particular length from the ultimate load of a reinforced steel column of the same length the additional load carried by the reinforced column by virtue of the concrete core may be computed. It will be of interest to compare this load, or rather the unit stress on the core concrete computed from this load, with the strength shown by the same concrete in the cube and cylinder tests. In the

general summary sheet (folded insert) the items are exhibited in parallel columns and the ratios of the column stress to the cube and to the cylinder stresses are also given. These results are plotted in the diagram on page 85 for all the core section columns of all mixtures. The comparison between the cube and the column strength seems to be well expressed by a straight-line relation, indicating that the column concrete develops roughly two-thirds of the strength of the same concrete tested as 6-inch cubes. The relation between cylinder and column strength seems more uncertain, being best represented by a curved line. For the region including the 1:2:4 mixtures the ratio seems to be about one, while for 1:1:2 mixture the cylinder shows higher strength, and for the 1:3:6 mixture the column shows higher. An average of all the ratios given on the summary sheet is about one. All these ratios are based on the strength at sixty days under rather poor conditions of curing.

28 - Values of E and n. In the general summary sheet the values of the secant moduli of elasticity for the concrete item have been tabulated for unit-deformations of .0004, .0007, and .0010, and for a deformation near the ultimate. It is interesting to compare these values with the values of E for the steel and to determine the ratio of the stresses in the concrete and in the steel at these unit-deformations. It is recognized that values from these concrete curves do not represent with absolute fidelity the conditions actually existing in the column but they do give an adequate idea for design purposes. The ratios are


Ultimate Strength of Cylinders
#1/0"
Reinforced Concrete Columns
4'-8"; 10'-0"; 15'-4"; & 19'-4" Long
Core Sections Only
Comparison of Strength of
Cubes and Cylinders with
Strength Developed in Columns
3000
2000
1000
0
Col. = 0.68 Cube
Ultimate Strength Developed in Columns
Ultimate Strength of Cubes - Lbs./Sq. In.


Reinforced Concrete Columns
4'-8", 10'-0", 15'-4" & 19'-4" Long
Core Sections Only
Comparison of Strength of Cubes and Cylinders with Strength Developed in Columns
Ultimate Strength of Cylinders
Ultimate Strength of Cubes - lbs./sq.in.
Ultimate Strength Developed in Columns
3000
2000
1000
0

also tabulated and these values of n afford a very interesting study. It is to be noted first of all that the value of n is larger than we ordinarily consider it to be, due largely to the fact that the concrete in the column is less dense than the same concrete would be if allowed to compact freely. A second striking fact is the exceedingly limited variation in the value of n for widely different values of the unit-deformation. For the 1:2:4 mixture where the cube strength varied 30 per cent from the mean there is also found a large variation in the value of n for different columns. These 1:2:4 columns are divisible into two groups of nearly the same quality of concrete. Columns Nos. 8908, 8913, 8918, and 8923 showed a variation in the 90-day cube strength of less than 2 per cent from the average, although the variation in the 60-day cube tests was much greater. Columns Nos. 8907, 8912, 8917, and 8922 form the other 1:2:4 group and exhibit a much wider variation in the cube and cylinder strength. The following table gives the values for n for the four groups into which the columns tested seem to fall naturally.

Table of Values of n.

Mixture	Crushing Strength at 60 Days			Average Value of n
	Cube	Cylinder	Column	
1:3:6	1400	690	990	38
1:2:4	1500	1150	1050	33
	2200	1300	1450	23
1:1:2	3100	2475	2125	16

In considering the above valuesit must be borne in mind that the concrete was not as well seasoned as would ordinarily be the case in building construction. It would seem that for 1:2:4 concrete a value of n = 20 would be a reasonable one to use in design, permitting the use of structural steel to much better advantage. The table below gives a summary of the values of n for various mixtures which the results of this series of tests seem to indicáte as reasonable, compared with the values permitted by the new Chicago building ordinance.

Table of Recommended Values of n.

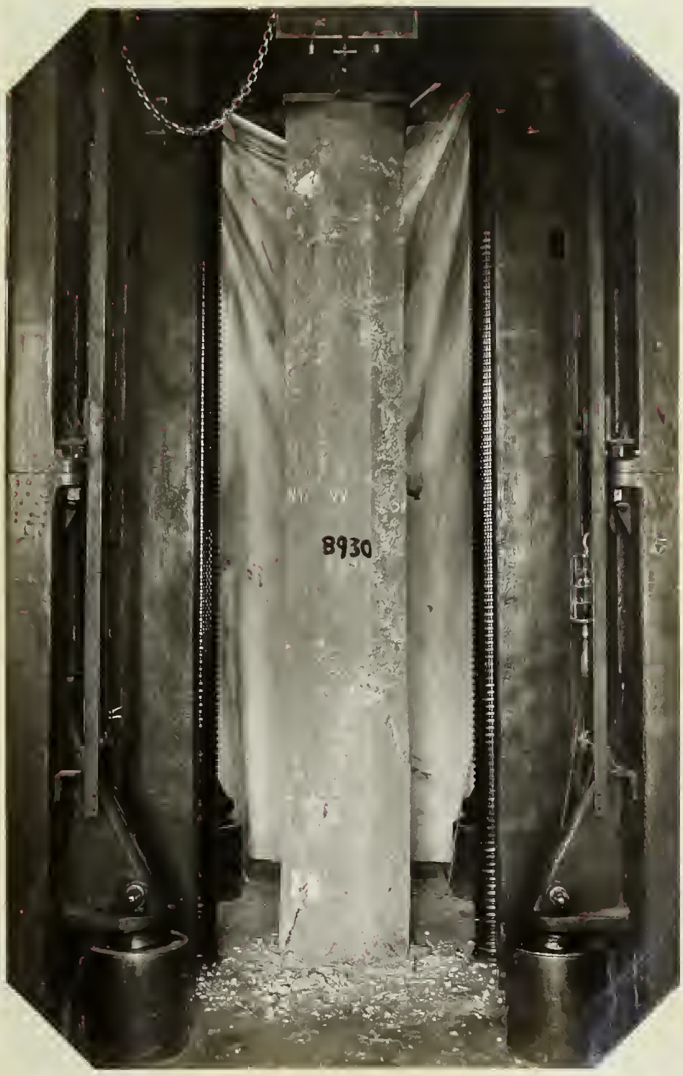
Mixture	Recommended Value	Chicago Code
1:1:2	12	10
1:1 1/2:3	15	12
1:2:4	20	15
1:3:6	30	20

29 – Published Conclusions on the Strengthening Effect of Concrete. The series of tests at other laboratories will be treated in some detail at a later time, but at this stage it will be interesting to simply state some of their conclusions. Dr. von Emperger decided from his investigation that the concrete could be relied upon to develop its full compressive strength in the column (cf. paragraphs 27 and 41) Professor Withey states that his tests show the concrete column to possess nearly the strength of the plain steel column plus the strength of the plain concrete column of the same area as the core of the concreted column (cf. par. 40).

Professor Withey's conclusions interpreted in terms of the cylinder strength are practically identical with those deduced from this series of tests and stated in paragraph 27, although the steel percentage was much smaller in the Wisconsin tests.

FIREPROOFED COLUMN

INSTRUMENTS ATTACHED



COLUMN NO. 8 9 3 0

JUST BEYOND ULTIMATE LOAD

COLUMN NO. 8 9 3 0

SHOWING FINAL STRIPPING

COLUMN NO. 8 9 3 1

SHOWING FINAL FAILURE

D REINFORCED STEEL COLUMNS - FIREPROOFING

30 - Phenomena of the Tests. The columns encased in a 2-inch coat of concrete outside the steel and the core section were tested to determined the additional strength afforded by this covering and to study the behavior of this fireproofing coat under load. During the earlier stages of the test there was no difference between the behavior of this type of column and one of core section except for the natural refusal of the outer shell to take load as rapidly as the core concrete. The concrete shell remained intact until the ultimate deformation of the column was practically reached, the unit-deformation being .0018 or over at first crack. The vertical steel was evidently starting to yield and the concrete in the core was very close to its ultimate. When the shell cracked the total load on the column dropped off about 65 000 pounds and the strength of the core was well evidenced by the long time during which the load remained at this second ultimate, the machine being in operation in the meantime. The load rose 2 000 to 4000 pounds after its first drop, showing that the strength of the enclosed concrete and of the vertical steel was not quite fully developed when the shell cracked.

31 – Stress-deformation Relations. Owing to the presence of the shell, constituting 47 per cent of the total concrete section, and which could not be expected to carry its full share of the load, the value of the modulus of elasticity is somewhat less for the fireproofed columns than for the core sections. Where n for the latter averages about 23 it becomes more nearly 30 for the former. Since it is undoubtedly wisest not to figure on any load on the shell in designing this value of n is significant only in showing that the use of the values of n recommended in paragraph 28 does not threaten the safety and integrity of the shell.

32 – Comparison of Concrete Stresses on Gross and Core Sections. Of the three fireproofed columns tested two only were crushed. Column No. 8930 took a maximum load of 630 700 pounds, which fell to 565 000 pounds when the shell failed, showing a load of 65 700 pounds carried by the shell. For column No. 8931 the maximum load was 635 700 pounds, the core load was 572 000 pounds, leaving 63 700 pounds on the shell. Of the core load we may consider 418 000 pounds as carried by the steel column, leaving on the core concrete 147 000 pounds for column No. 8930 and 154 000 pounds for column No. 8931. The area of the shell was 93 square inches and of the core concrete 107 square inches and hence the unit-stresses are as follows:

Concrete Stress on Shell and on Core.

	Unit Concrete Stress on — Gross concrete	Shell concrete	Core Conc.
Column No. 8930	1063	706	1374
Column No. 8931	1088	685	1440

It may be said then that the concrete of the shell carries roughly one-half as much load in pounds per square inch as the concrete confined within the core.

33 – Permissible Deformation as Governing Design. The question always arises in design as to the effect of the deformation of the column under load on the integrity of the shell. If the shell goes to pieces at relatively low unit-deformations it is evident that deformation rather than stress must govern in the selection of the working stresses to be used in design. As noted in paragraph 30 the shell did not crack nor the load drop off until the unit-deformation exceeded .0018 and this figure is practically the ultimate for a condition of increasing load on the core section also. In other words, the action of the shell during the test did not seem to be such as to impose any restriction on the selection of the working stresses.

34 – Need of a Tie to Prevent Stripping of Shell. While it has been stated above that the shell maintains itself intact under high unit-deformations it still seems to the writer that a tie of some sort (wire mesh, spiral, or other binder) should be imbedded in the outer shell to make its condition certain. The backs of the flanges occupied a

quarter of the bonding surface between the shell and the core, and in practice this proportion might easily be greater. It seems unsafe to trust the shell to stand uninjured by collisions and accidents with so large a surface so uncertainly supported. This tie would also prevent the rapid and complete failure of the shell at maximum load which occurred in the test columns, although this advantage in not very great in actual construction. If a spiral is used for strength, which seems to the writer a somewhat unwise expedient, it might serve to increase the ultimate unit-deformation of the shell but probably not to any large degree. A light spiral, used simply as a precaution against accidental damage, is recommended for a tie in the fireproofing shell of reinforced steel columns.

TYPICAL VIEW OF

SPIRALLED COLUMNS

E REINFORCED STEEL COLUMNS WITH SPIRALS

35 – Phenomena of Tests and Stress-deformation Relations. For a loading within the capacity of the University of Illinois testing machine (600 000 pounds) the load-deformation curves of the columns with three-quarters per cent and with one per cent of spiral reinforcement are practically identical in nature with those for core section only. Within the limits of this first test the action of the column is in no essential particular different from that of core section columns. As was mentioned in paragraph 9 it was found necessary to grout the space between the spirals and the back of the flanges with cement mortar. During the test this filling was observed to behave in a special way. Under high loads it cracked vertically (over backs of flanges only, where the rich mortar was), these cracks opening quite wide at 600 000 pounds load. When the load was released the vertical cracks closed so as to be hardly visible and horizontal cracks opened at intervals of from six to eight inches and extended the width of the flange very nearly. This peculiar phenomenon was probably due to the small capacity of the mortar to withstand deformation and to the large set present in such mortar when it has once been highly stressed. After standing in an unloaded position for several weeks the

vertical cracks were found to be more noticeable than the horizontal ones, which latter had closed almost entirely.

Four of the spiraled columns were subsequently tested a second time at Lehigh University where a load of 830 000 pounds was applied. The logs of these second tests are tabulated on pages 215 to 220, and the stress-deformation relations are shown on the diagrams, pages 152 to 157, on which diagrams the stress-deformation curve for the first test of the same column is given in each case as a broken line. It must be borne in mind that the columns were roughly three months older at the second test than when first tested, and the increased strength of the concrete may account for a portion of the increased load carried in the second test. To secure some conception of the amount of this added strength due to the greater age of the concrete column No. 8937 was tested a third time after the spiral had been removed and the column reduced to the standard core section. The log of this test (see page 219) shows an ultimate load for this column of 714 000 pounds, showing that a considerable portion of the added load carried in the second test was due to the greater age of the concrete.

The phenomena of the tests were not essentially different from those described above under the first tests. At a load of about 750 000 spalling began on the columns with 3/4 per cent spiral reinforcement, and this spalling continued during the remainder of the test. It seemed that these columns were very close to their ultimate load at the end of

the test. With the columns with 1 per cent spiral reinforcement the spalling was extremely slight, and these columns would evidently have carried considerable more load. An examination of the enclosed structural steel column showed no crimping of the flanges at the end of the second tests, and no movement of the parts relative to one another. The spiral seems to have acted to restrain this steel column and to cause it to take greater deformations and higher load than it normally would.

36 - Effect of Different Percentages of Spiral at Working Loads. Two different percentages of spiral reinforcement were used in the columns tested. Columns Nos. 8933 to 8936 had three-fourths of one per cent, while Nos. 8937 and 8938 had one per cent. Within the limits of the first test (600 000 pounds load) the load-deformation curves do not show any effect which can consistently be attributed to the variation in the spiral percentages. For these load-deformation curves see pages 146 to 151.

37 - Effect of Different Percentages of Spiral at Ultimate Loads. Under paragraph 35 the fact that spalling began earlier with columns with 3/4 per cent spiral than with columns with 1 per cent spiral has been mentioned. In general the maximum applied load evidently approached very close to the ultimate for the 3/4 per cent columns, while it did not stress the 1 per cent columns nearly as severely. The stress-deformation curves (pages 152 to 157) show that the yielding takes place at practically the

same load for all percentages.(650 000 to 700 000 pounds), but that after this yield, when the spiral begins to play an important part, the columns with the heavier spiral show a greater stiffness and strength. Since the tests were not carried to destruction the amount of this added strength cannot be ascertained, but its existence seems well established. Judging from the stress-deformation curves the amount is not relatively very great for any of the percentages of spiral used. For most purposes the significant point on the stress-deformation curves is the point where the curve bends sharply and becomes very flat. Beyond this point any added load is carried only by virtue of very greatly increased deformations. For the four columns tested this point lies at or below 750 000 pounds load and it will be interesting to compare this with the strength of the column without spiral reinforcement. For the purpose of this comparison column No. 8937 was stripped of its spiral and reduced to the standard core section. This column carried 714 000 pounds load as against 580 000 pounds carried by core section columns at 60 days age. If we consider all the concrete within the spiral in the second tests to be as effective as this core concrete in column No. 8937 the load carried without aid from the spiral would be about 800 000 pounds. This figure is probably a little high as the concrete outside the core proper, although inside the spiral, would probably take some less load per square inch than the concrete in the core. From this it may be observed that the strength of the column not counting in the

spiral is equal to the strength of the column tested with the spiral at the point when the stress-deformation diagram shows its final yield, and that only the strength developed after this yield is passed should be attributed to the spiral directly. In column No. 8934 this added load amounts to about 100 000 pounds, which is 715 pounds per square inch of enclosed concrete. This is at the rate of 1000 pounds per square inch of core concrete per one per cent of spiral reinforcement. Applying this figure to the 1 per cent spiraled columns the computed maximum load becomes 900 000 pounds, a value which the deformations of the second tests seem to indicate as a reasonable expectation for these columns.

38 - Availability of Spiral Strength for Design. The tests of this series of columns show very conclusively that up to a unit-deformation .0015 there is not found any appreciable difference in the action of columns with and without spiral reinforcement. In building construction the safe limit of unit-deformation may ordinarily be placed at .0005 or less. It would seem, then, that any attempt to use an imaginary spiral strength at working loads could only result in an actual unit-deformation much greater than .0005 with resulting danger to other parts of the structure. The practise of many designers, based on the conclusions of M. Considere, in which the volume of the lateral steel is figured and a high stress is computed on imaginary verticals of 2.4 times this volume, has been shown to be illogical

and dangerous for reinforced concrete columns by the tests at Illinois, Wisconsin, and elsewhere. The spiral does afford protection against sudden failure, gives a tougher and safer column, and so may rightly be considered a warrant for the use of higher unit-stresses in spirally reinforced concrete columns. In the reinforced steel columns tested in this thesis investigation the need of a spiral is much less than in the reinforced concrete column. The steel column possesses much of the toughness of the spiraled column and the flanges of the angles tend to restrain the core concrete to some extent, so that most of the really valuable features of the spiraled column are already assured in the reinforced steel column. The failure is very gradual and the unit-deformation at the ultimate load is quite large without any spiral present. Thus it appears to the writer that the use of any large percentage of spiral reinforcement with reinforced steel columns is unjustifiable. A light spiral may well serve to tie the shell together and protect it from accident, but should not be directly computed for strength. It is probably permissible to consider the entire section within the spiral as the core, and the chief function of the spiral as an element of strength seems to the writer to consist of its action to make the concrete outside the true core and within the spiral take load equally with the concrete in the true core. This condition was very nearly satisfied in the columns tested.

F COMPARISON OF RESULTS WITH OTHER TESTS

39 - University of Illinois Tests - Series of 1908. The table on the following page gives a list of the specimens tested at the University of Illinois in 1908, and a summary of the more important data secured from the tests. For the number of variables used the number of columns was entirely inadequate, and the result has been that the impressions of the series are somewhat confusing and uncertain. The series also suffers in that no plain steel columns were tested to afford a basis of comparison. The presence of relatively large shell sections acts to reduce their apparent strength somewhat (cf. paragraph 32), and the ratio of the shell section to the core section is not constant. The load-deformation diagrams are given on pages 158 to 167, the line for steel on these diagrams being determined from the tests of the plain steel columns of the Gray type tested as a part of this thesis by a reduction of load in the proportion of the areas of the columns.

It is to be noted from the table that the ratio of strength developed by the concrete in the columns to that shown by tests of 6-inch cubes was about 55 per cent on the

average, the results showing the wide variation of from 40 to 70 per cent. From the diagrams a marked tendency may be noted for the concrete curve to reach a maximum and decline before the column as a whole reaches its maximum load. This tendency is not shown in the tests described in this thesis proper, although the increase at high loads is relatively small (cf. paragraph 26).

Summary of Results

University of Illinois Tests - Series of 1908

Column No.	Steel Present	Area of Steel	% Steel	Ratio Core to Gross Section.	Failure
31°	4- 3x3x1/4 in. Ls	5.76	3.0	70.0	Slow
48	12 in. b.b. Laced				
32°	4- 3x3x1/2 in. Ls	11.16	6.0	68.0	Slow
52	12 in. b.b. Laced				
34°	4- 3x3x1/2 in. Ls	11.16	6.0	68.0	Hooped
56	12 in. b.b. Tied				effect.
40	4- 2x2x3/8 in. Ls	5.76	3.0	38.0	Sudden
61	9 in. b. b. Laced				
45	4- 3x3x1/2 in. Ls	11.16	6.0	36.0	Sudden
63	9 in. b. b. Tied				

All columns were 10 feet long and were concrete to a 14 x 14 inch section.

° These columns were tested with 1 in. concrete between steel columns and bearing plate at upper end.

Summary of Results

University of Illinois Tests - Series of 1908

Continued.

Column No.	Max. Column Load	Unit Load on Column	Age Col. Da.	Concrete Strength Col.	Col./Cube	Cube	Age Cube Days	Maximum Unit-Deform.
31	438 000	2300	63	1441	.56	2570	70	.0013
48	440 000	2310	65	1416	.70	2025	74	
32	400 000	2100	63	975	.46	2125	69	.00085
52	444 000	2400	61	758	.41	1844	64	
34	570 000	3080	53	1200	.64	1885	63	
56	480 000	2600	59	675		2680	59	
40	419 000	2200	64	1300				.00145
61	390 000	2040	59	1100	.61	1820	61	
45	495 000	2670	65	1175	.47	2460	84	.0013
63	434 000	2340	60	605	.55	1110	64	

40 – University of Wisconsin Tests – Series of 1908.

This series consisted of five columns built up of 2 x 2 x 3/16 inch angles 8 inches back to back, latticed with 1 3/4 x 3/16 inch bars, and tied with batten plates at top and bottom. One column was tested plain, two with the core filled with concrete, and two with fireproofing to make a 12 x 12 in. column. A column of plain concrete of the same area as the core of the column of core section was also made and tested at the same age. The table on the following page gives the data of the tests and the diagrams on pages 169 to 172 give the load-deformation relations.

In this table the load carried by the shell should be noted especially. It averages 720 pounds per square inch which is practically the same as that found in paragraph 32. The value of n is much lower than that given for the thesis columns, due in part to the lower percentage of reinforcement which offered less obstruction to settlement, and in part to the higher modulus of the concrete as shown by the cylinder tests. The ratio between the strength of the concrete in the column and that in the cylinder is .715 and .775 against a ratio of .67 for the cubes in the thesis tests. Since the cylinders were 6-in. diam x 8-in. long they would be expected to show a little less strength than a 6-in. cube, and the agreement between the two series seems very close in this respect. In general the agreement between the Wisconsin tests with only 4.5 per cent of structural steel and the reinforced steel columns with 10.8 per cent, is striking, and

would seem to indicate that the variation for different types of columns and for different percentages is not so large as is sometimes considered. The Wisconsin tests all show a decrease in the concrete load before the maximum for the column as a whole has been reached.

Summary of Results

University of Wisconsin Tests - Series of 1908

Column No.	Per Cent Steel	Ratio Core T.Area	Load Carried Total	Load Carried Unit	U.Load on Concr.	U.Load on Cyldr.	Ratio Col. Conc. to Cyl.
B 1	4.5	95.5	237 000	3700	1620	2270	.715
4	4.5	95.5	242 000	3780	1700	2200	.775
2	2.0	42.5	303 000	2100	1170	2640	.445
3	2.0	42.5	288 000	2000	1065	2140	.500
BS	Plain Steel		138 000	48000			
F 1	Plain Concrete		120 600	1880	1880	2250	.835

Column No.	Unit Load on Shell	Unit Def. Cracking of Shell	Value of n .0004 Unit Def.
B 1			9.1
4			9.6
2	812	.0015	11.2
3	625	.0015	12.1

41 – Tests by Dr. von Emperger. This is the only series of tests so far published which give results for high percentages of vertical steel in the shape of structural columns. The sizes of the columns were very small. All the columns had once been tested as plain steel columns before being reinforced with concrete. The data concerning the concrete or mortar used are very meagre, the cubes apparently being tested at different ages from the columns and no record being offered as to the cube ages, so that is hard to know how far the cube strengths may justly be compared with the column strengths. The fact that in many cases the concrete in the columns apparently takes a load from 20 to 70 per cent greater than its ultimate when tested as 8-in. cubes seems to the writer to indicate a need of more information. A number of columns (A5, B2, B5, B6, C1, and D1) check the results of this thesis very closely. Others indicate that the concrete in the columns is taking a much greater proportinate load (compared to cube tests) than did the thesis columns. About one-half of the columns check the results of the Illinois and Wisconsin tests quite satisfactorily. The conclusion stated by Dr. von Emperger, that the concrete in the columns can be depended upon to develop its full cube strength, does not seem to the writer to be warranted when one-half of the columns tested fell considerably short of this. The data of the tests, converted into English measures, is given in the following table, and the notes following this table serve to more fully describe the specimens.

Summary of Results

Tests by Dr. von Emperger.

Column No.	Length	L/r for Steel	Unit Load Carried Reinforced Col.	Plain Column	Load on Conc.	Cube Strength Conc.	Ratio	Age
A1'	7'– 0"	37.3	9600	28 350	6150	5235	1.17	6 yrs
A2'	11 – 6	63.5	8740	16 800	7230	5235	1.38	6 "
A2	11 – 6	63.5	7010	16 800	5200			4 wks
A3	11 – 6	63.5	8840	29 500	5040	2965	1.70	8 "
A4	11 – 6	63.5	8580	37 150	3325	2390	1.39	8 "
A5	11 – 6	63.5	8610	41 500	2580	2955	.87	8 "
A6	17 – 8.5	95.2	7980	36 400	2775	2755	1.00	8 "
B1	11 – 6	65.2	10770	28 600	3120	2180	1.43	8 wks
B2	11 – 6	65.2	10200	29 350	1540	1955	.79	8 "
B3	1 – 7.7	9.6	13600	41 500	910	2970	.31	8 "
B4	3 – 3.4	19.2	13620	39 600	1845			8 "
B5	11 – 6	65.2	11780	34 550	1490	2290	.65	8 "
B6	11 – 6	65.2	12000	34 550	1795	2370	.76	8 "
C1'	6 – 11	24.8	5880	30 400	3290	5235	.63	6 yrs
D1'	3 – 5	18.3	6160	42 000	2120	5235	.41	6 yrs

Series	Col. Section	Sectional Area Steel	Concrete	% Steel	Rad. of Gyr. Steel
A	2 No. 14 Is	5.76	31.4	15.5	2.23
B	2 No. 14 [s	6.39	14.1	31.2	2.18
C	4 Ls	4.24	57.8	6.8	3.34
D	4 Ts	5.75	51.0	10.1	2.24

Notes on Test Specimens

No.	Notes
A	2 Is 5.51 x 2.99 in. Plates 9.2 in. long Is 5.22 in. on centers.
1	5 horiz. tie pls. 2.36 x .236 x 9.2 in. spaced 19.7 in. apart. Plates bolted to outer flanges of Is only.
2	Same with 8 plates on longer column.
3	6 pls. 4.72 x .236 x 9.2 in. 2 at each end and 3 sps at 39.4 in. 1 pl at end and one centered 10.8 in. from end.
4	8 pls do. 1 each end. end sps 21.7, int.sps. 19.7 in. 8 rivets per plate for (3), (4), and (6)
5	Latticed with 1.97 x .236 in bars - 1 pl as in (4) at ends
6	8 pls - 1 at ends - end sps 32.5, int. sps. 29.5 in.
B	2 [s 5.51 x 2.36 x .29 in. 2.56 in b.to b. 7.29 in.o.o.
1	6 tie pls. 4.72 x .236 x 7.29 in. 1 at end, 1 11.8 in. from end, and 3 sps at 39.4 in. 4 rivets per plate.
2	2 tie pls do. 1 at each end. 4 pls. 9.45 x .236 x 7.29 spaced 39.4 in. on centers. 8 rivets per plate.
3	2 tie pls as in (1) 1 at each end.
4	Same as (3)
5	8 pls as in (1), 1 at ends, and 7 sps at 19.68 in.
6	Latticed with 1.97 x .236 in bars - 1 plate at each end.
C	Ls 2.36 x 2.36 x .275 in. tied every 7.87 in. by 2.36 x .236 x 7.87 in. plates.
D	Ts 2.76 x 2.12 x .315 in. placed 7.22 in. b. to b. on sq. column with .843 in. chamfer on each corner when concreted. Tied straight across with 2 1.97 x .236 in. plates in each direction, spaced 10.8 in. on centers.

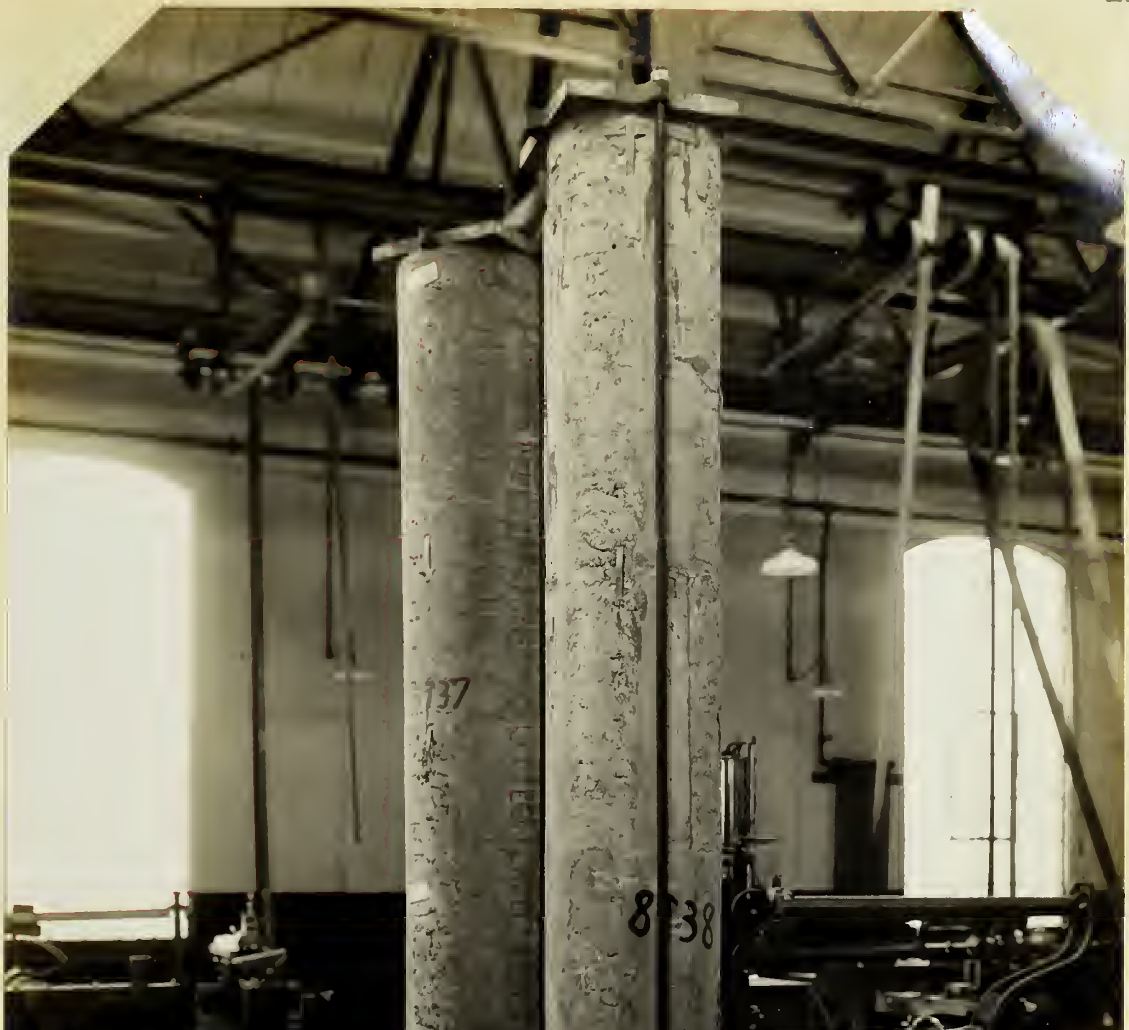
Concrete used was 1:3 mixture.

"TESTED" COLUMNS

TWO NOT CRUSHED


Col. No. 8934

Col. No. 8935



Cols. No. 8937 - 8938

IV

CONCLUSIONS

IV CONCLUSIONS

42 - Summary of Conclusions. The conclusions reached as a result of the study of the data of this series of tests may be briefly summarized from the discussion in the preceding paragraphs as follows. All these conclusions relate solely to the properties of the type and the sections of column tested and must be applied cautiously to other types and sizes of columns, although they may certainly be regarded as throwing light on the general questions involved in designing reinforced steel columns.

1 - The effect of slenderness on the plain steel columns at ultimate loads is best expressed by the straight-line formula

$$\frac{P}{A} = 36\,500 - 155\,\frac{L}{r}$$

2 - The effect of slenderness on the steel columns reinforced with core sections of 1:2:4 concrete and for ultimate loads is best expressed by the formula

$$\frac{P}{A} = 5\,100 - 45\,\frac{L}{d}$$

3 - The effect of slenderness is but slightly modified

by the presence of the core of concrete.

4 – The strengthening effect of the concrete within the core (or within the spiral, if such is present) is approximately equal to the full strength of the same concrete tested in the form of 8 x 16 in. cylinders, or to two-thirds the strength of the same concrete tested in the form of 6-inch cubes, of the same age as the columns.

5 – The fireproofing shell will remain intact to practically the ultimate deformation of the column, but will carry only about one-half as much load as the concrete confined within the core or spiral.

6 – A spiral has practically no effect on the load-carrying capacity of the column or on its stress-deformation relations up to a unit-deformation of .0015. The strength due to the spiral seems to be somewhat less available for design purposes than is the case with reinforced concrete columns.

7 – A moderate amount of spiral reinforcement may serve to warrant slightly higher stresses in the column, but must be used cautiously if at all. It seems preferable to assign to the spiral simply the function of rendering all the concrete inside its limits equal in resisting strength to that in the core proper, and of supplying protection to the outer shell against accidental damage or heavy pitting by fire.

8 – The value of n, the ratio between the secant modulus of elasticity of the structural steel column and that of the

concrete in the column core, may well be considered somewhat greater than the figures usually assumed. Recommended values of n for various mixes and for this type of column are:

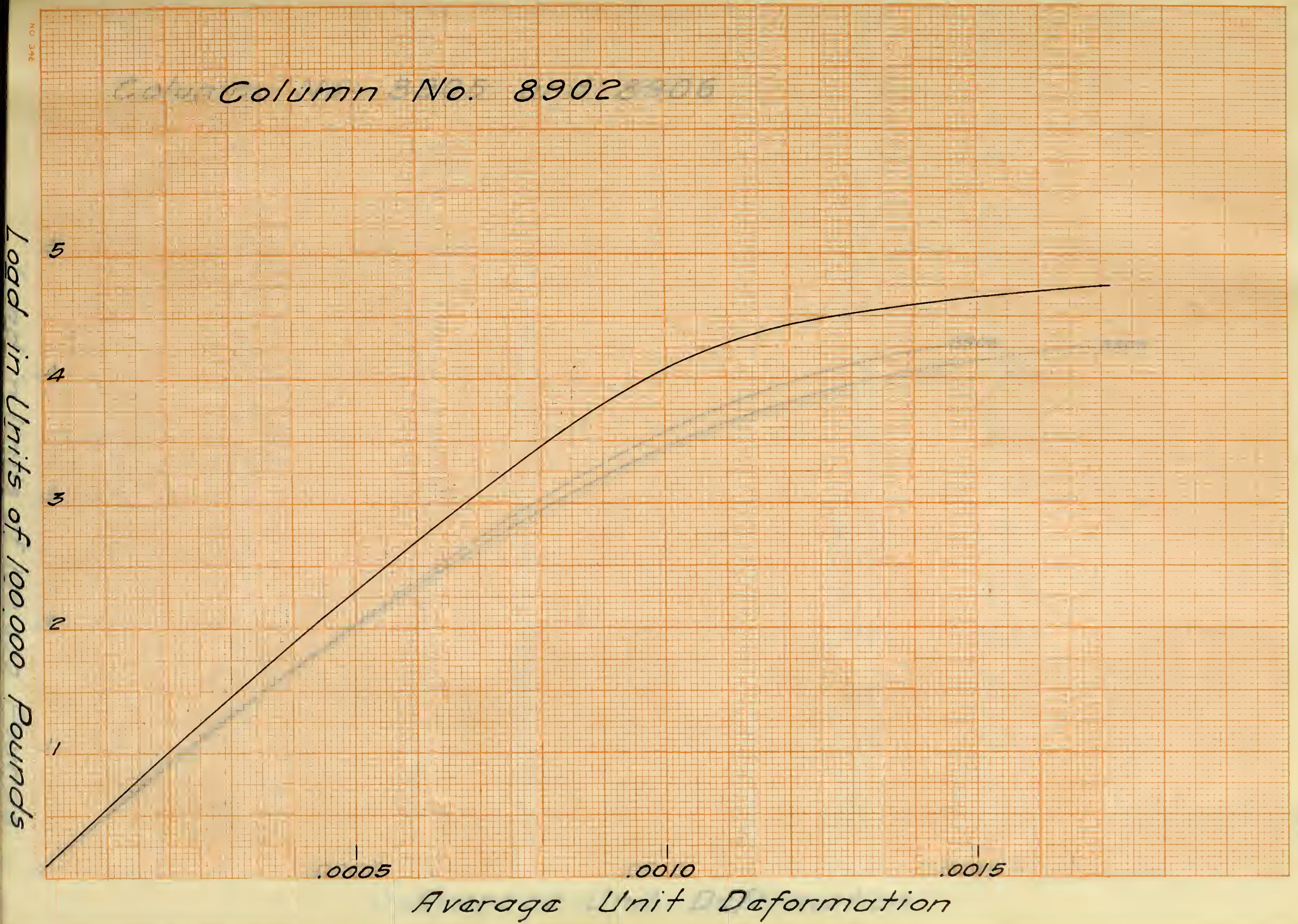
Mixture	Value of n
1 : 1 : 2	12
1 : 1 1/2 : 3	15
1 : 2 : 4	20
1 : 3 : 6	30

9 – In general these conclusions are well borne out for somewhat different conditions and test specimens by the previous tests so far published, which were not of themselves extensive enough to establish any well defined relations.

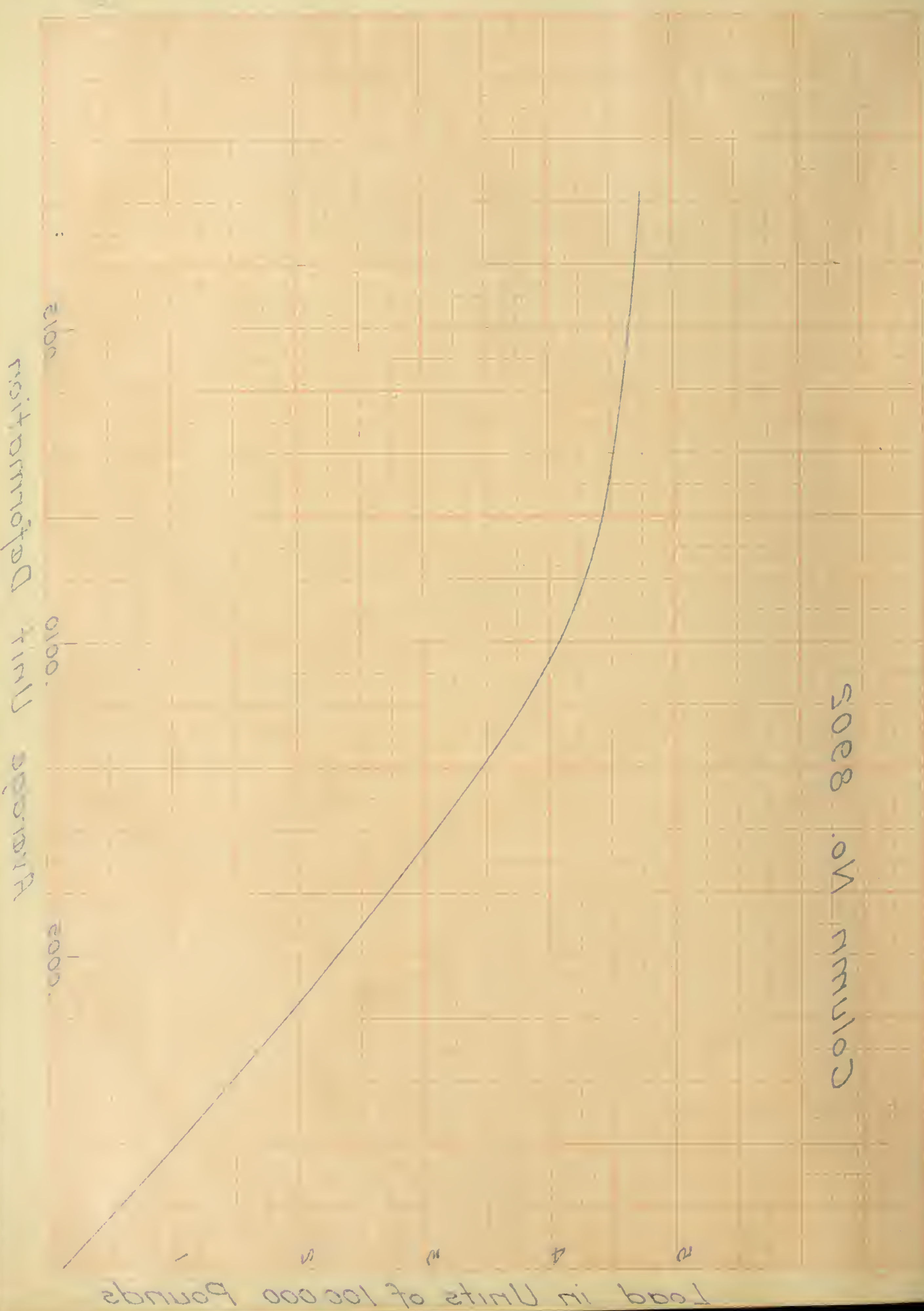
10 – The reinforced steel column of the type tested in this thesis investigation possesses the qualities of a first class structural member, and appeals to the writer as well adapted to much more extensive use in building construction than it at present enjoys.

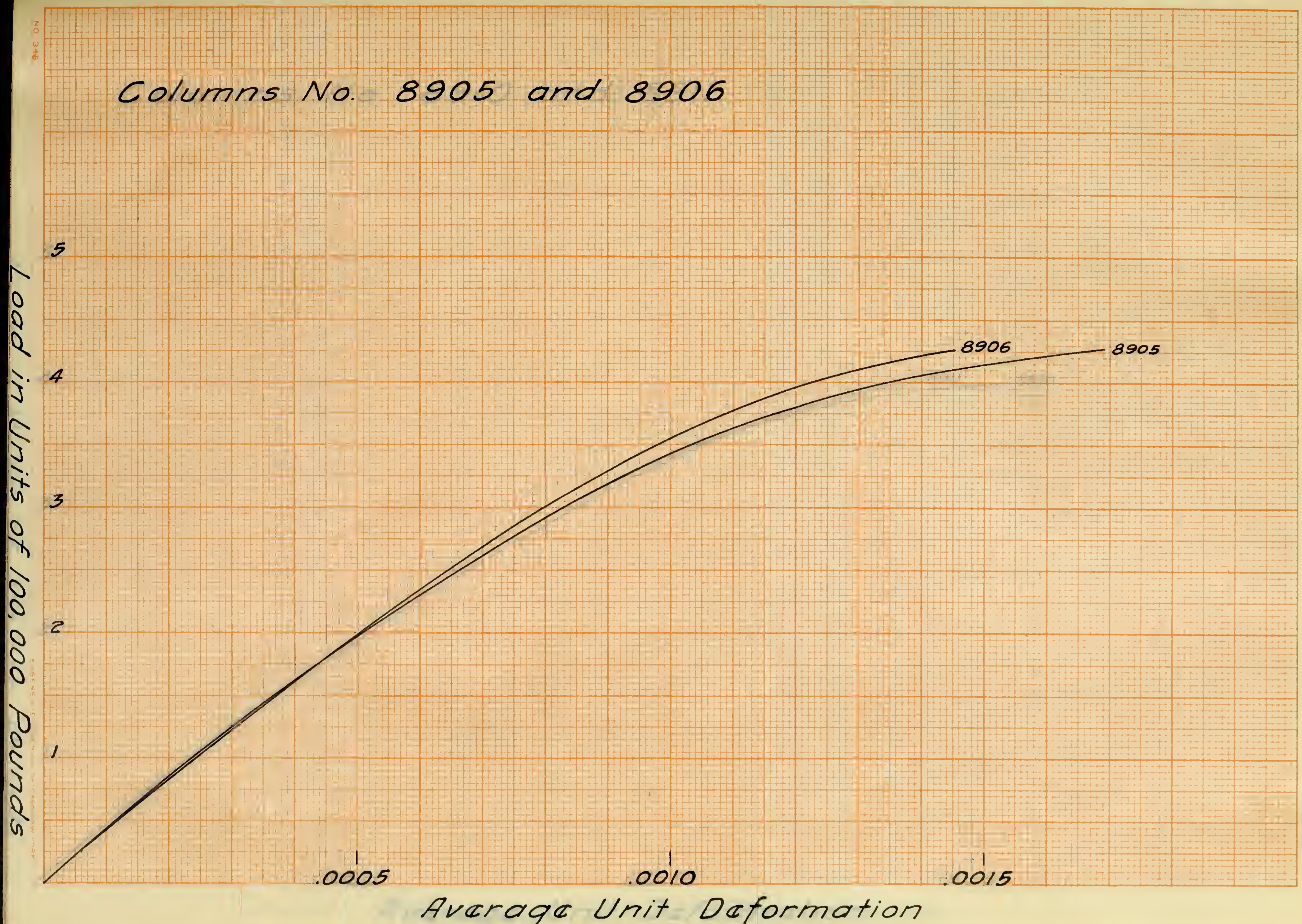
V

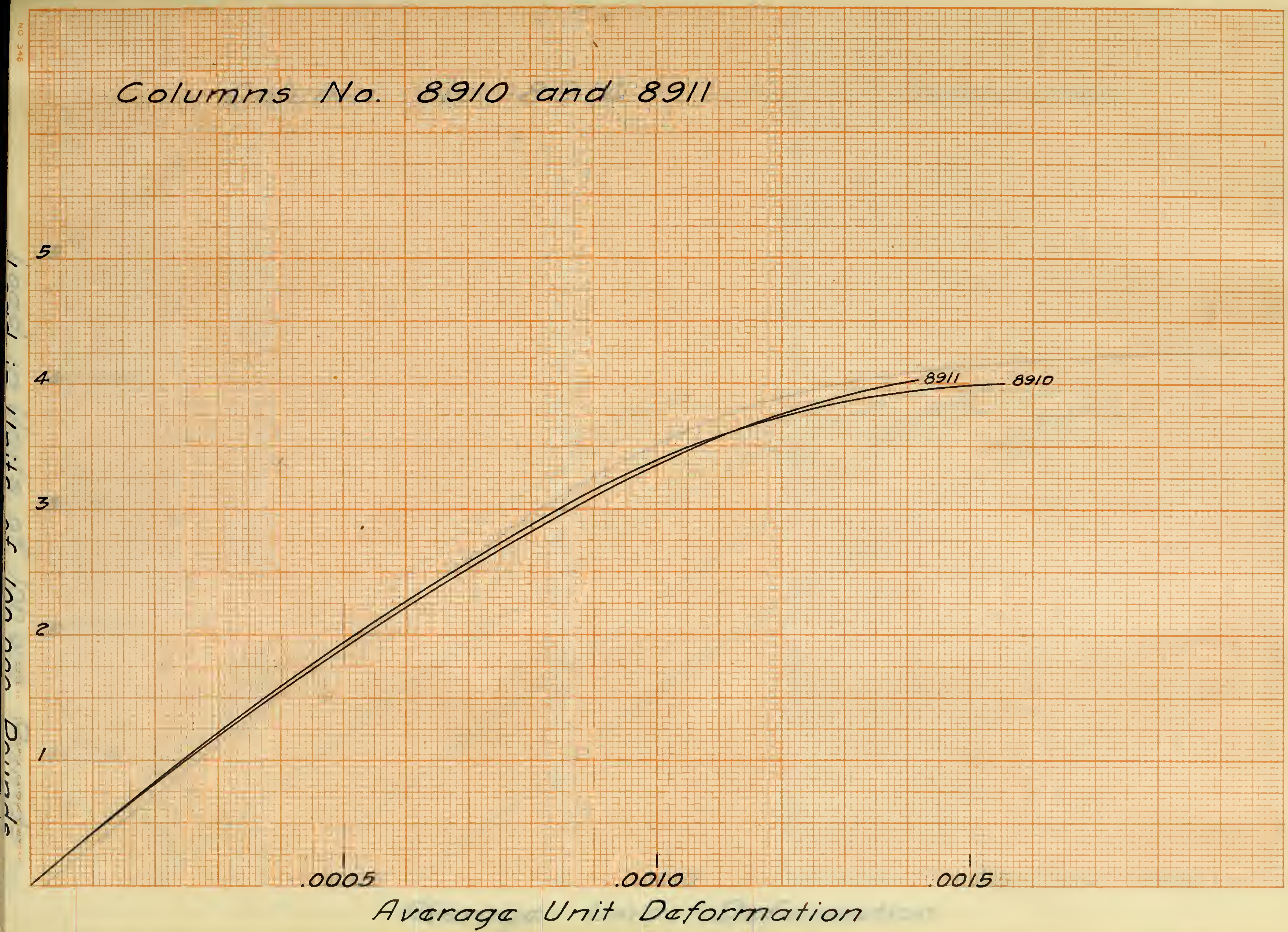
DIAGRAMS



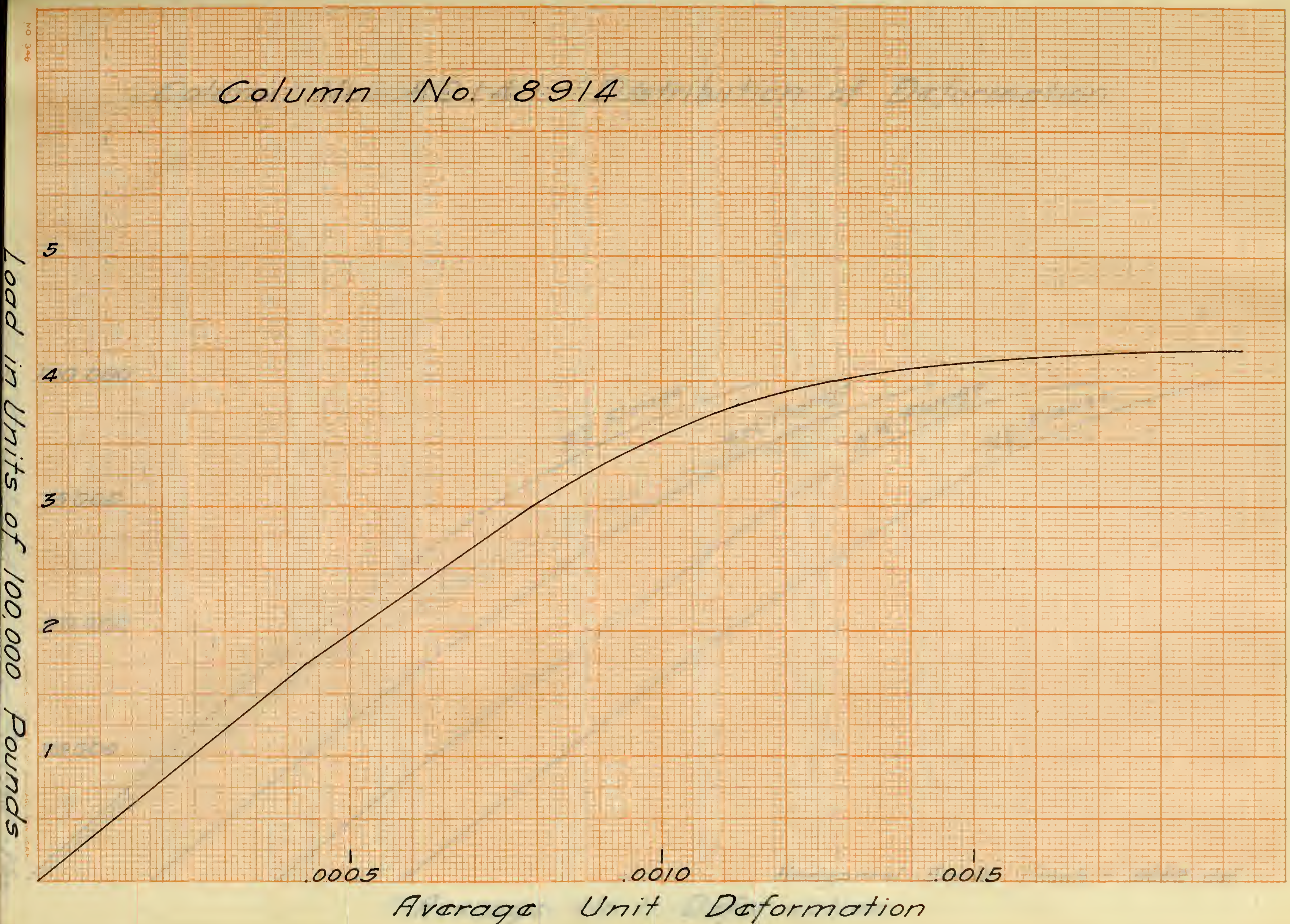
Column No. 8902
5
4
3
2
1
.0005
.0010
.0015
Load in Units of 100,000 Pounds
Average Unit Deformation

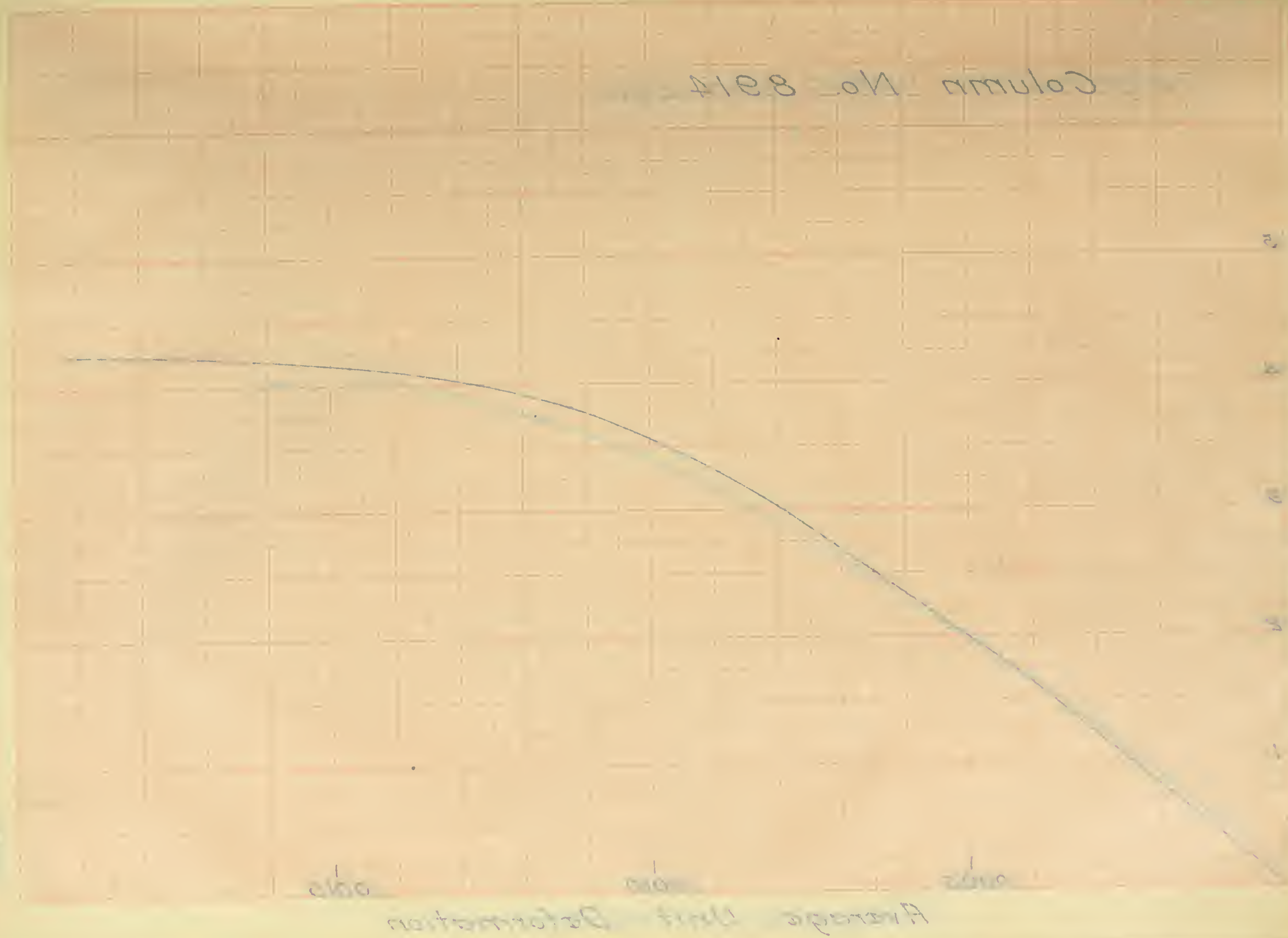

Column No. 8902
Average Unit Deformation
.0005
.0010
.0015
Load in Units of 100 000 Pounds
1
2
3
4
5


Columns No. 8905 and 8906
8906
8905
5
4
3
2
1
.0005
.0010
.0015
Average Unit Deformation
Load in Units of 100,000 Pounds

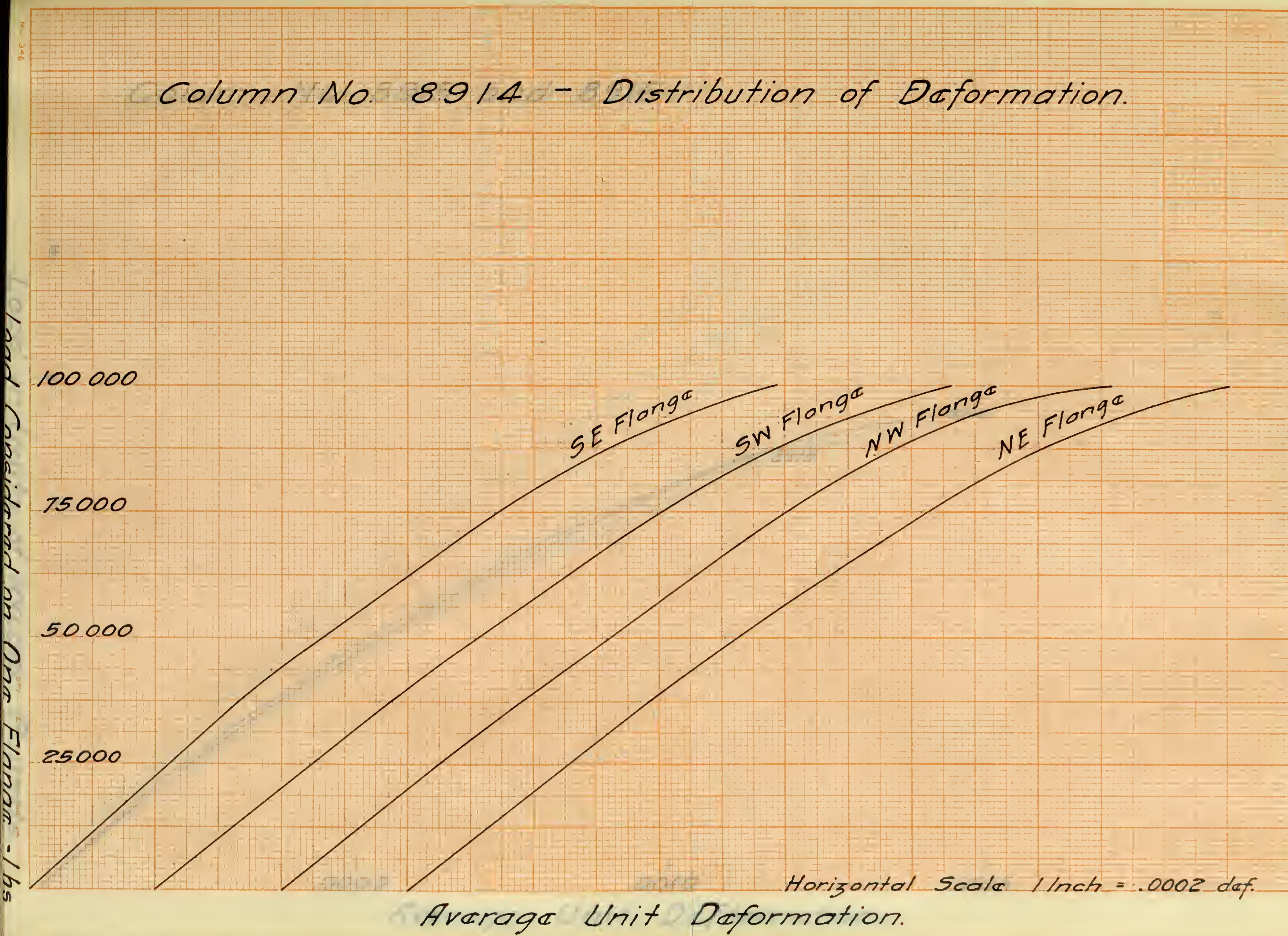

Columns No. 8910 and 8911
5
4
3
2
1
8911
8910
.0005
.0010
.0015
Average Unit Deformation

Column No. 8914

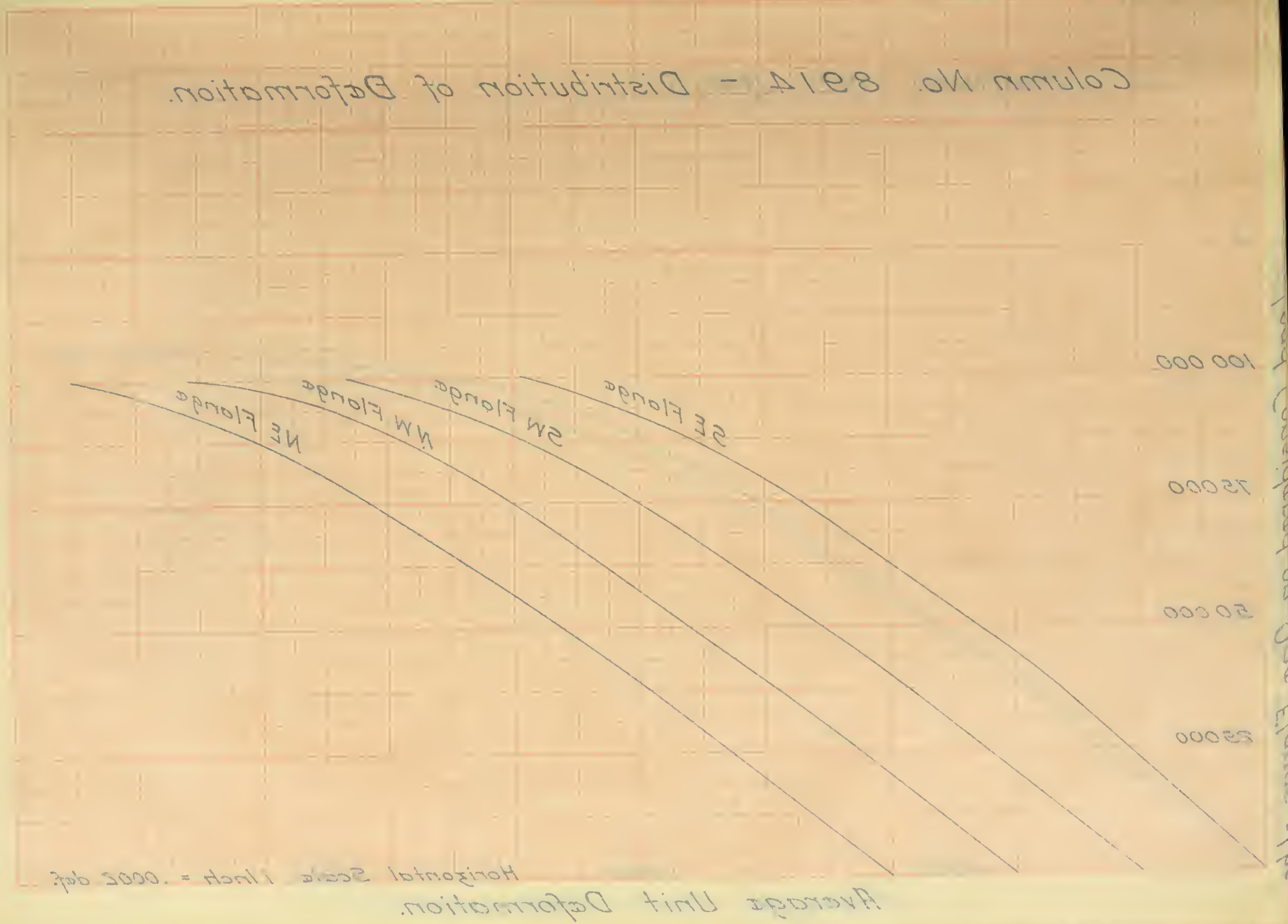




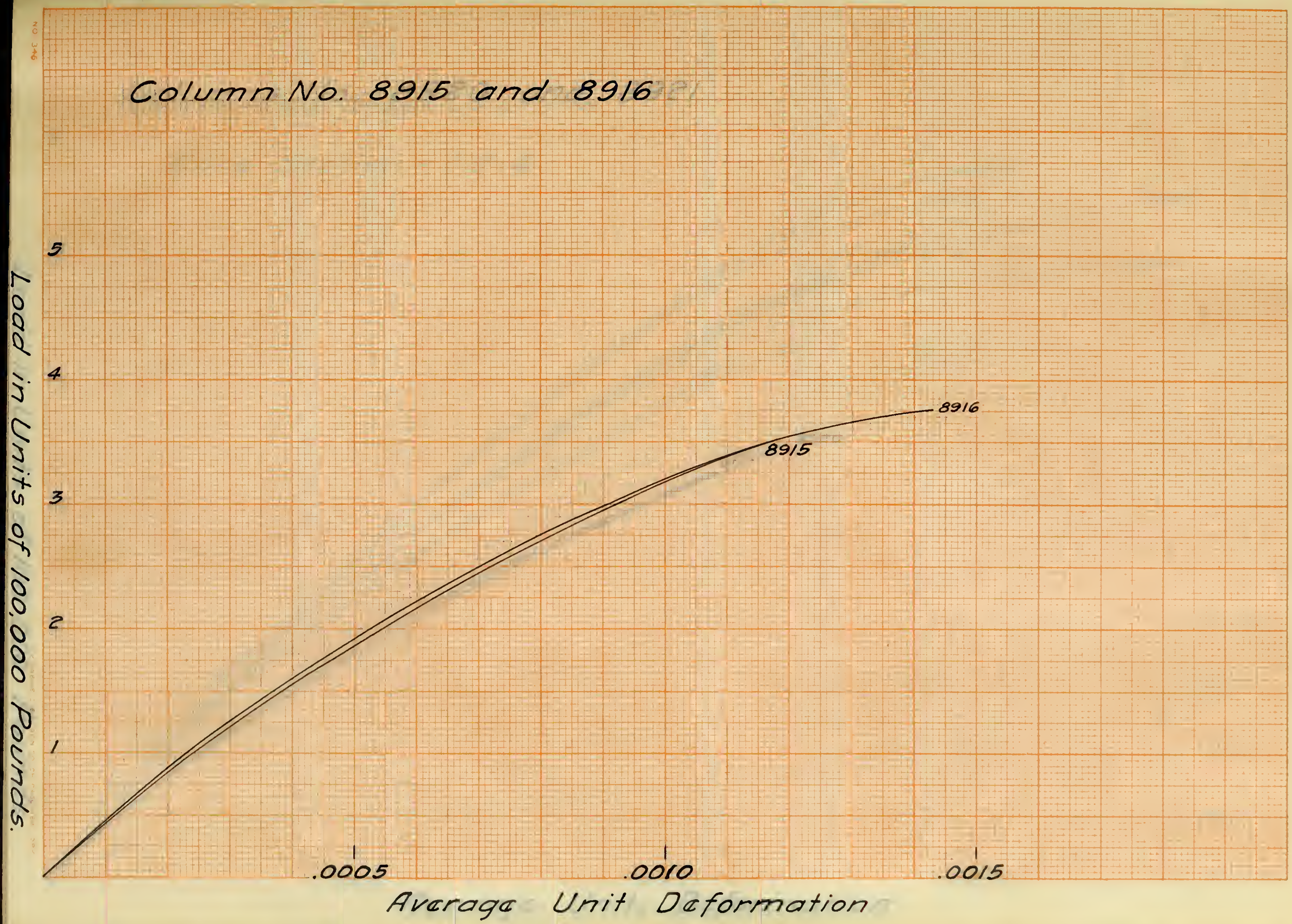
Column No. 8914
Load in Units of 100 000 Pounds
Average Unit Deformation



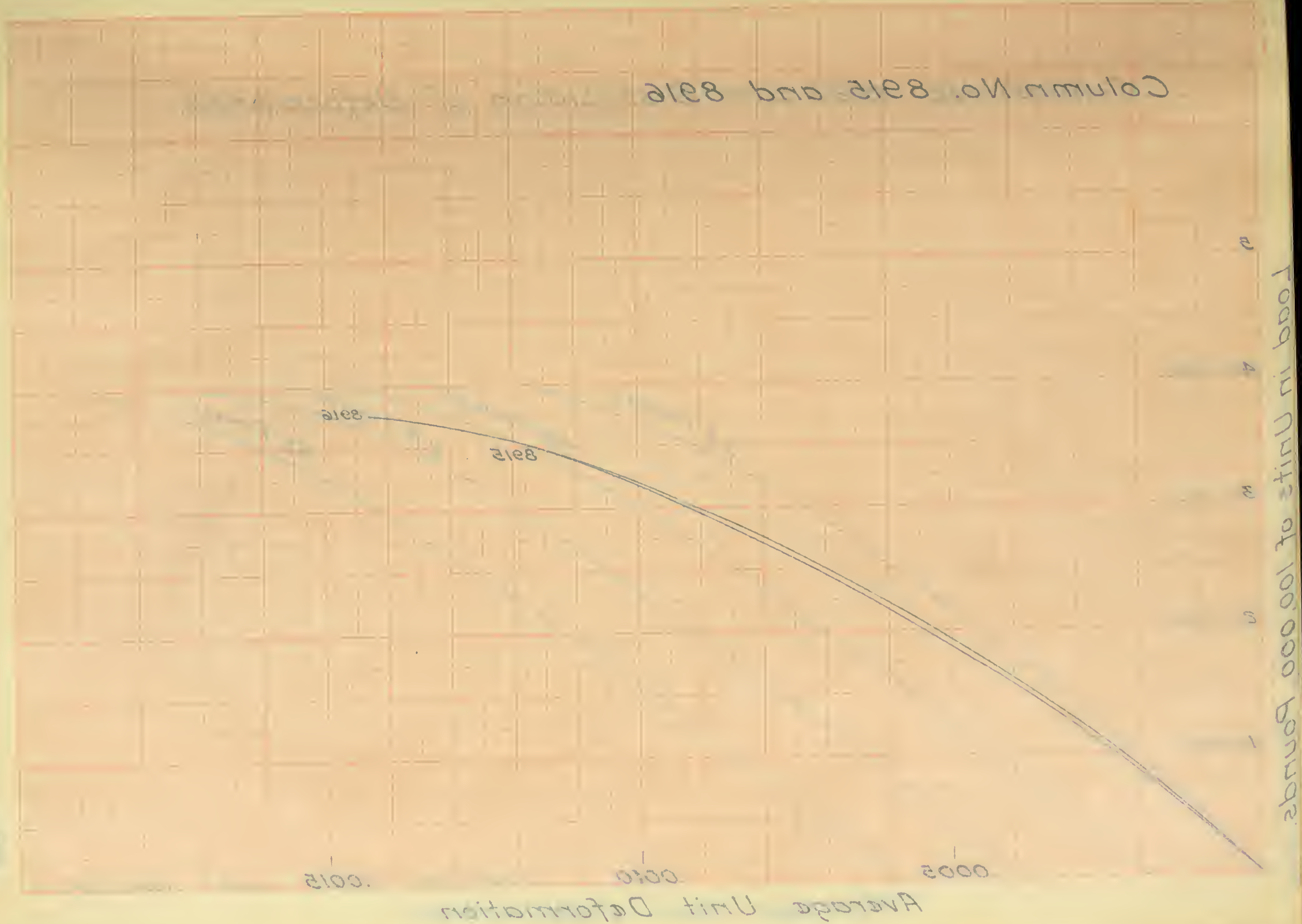
Column No. 8914 - Distribution of Deformation.
100.000
75000
50000
25000
SE Flange
SW Flange
NW Flange
NE Flange
Horizontal Scale 1 Inch = .0002 def.
Average Unit Deformation.
Load Considered on One Flange - lbs



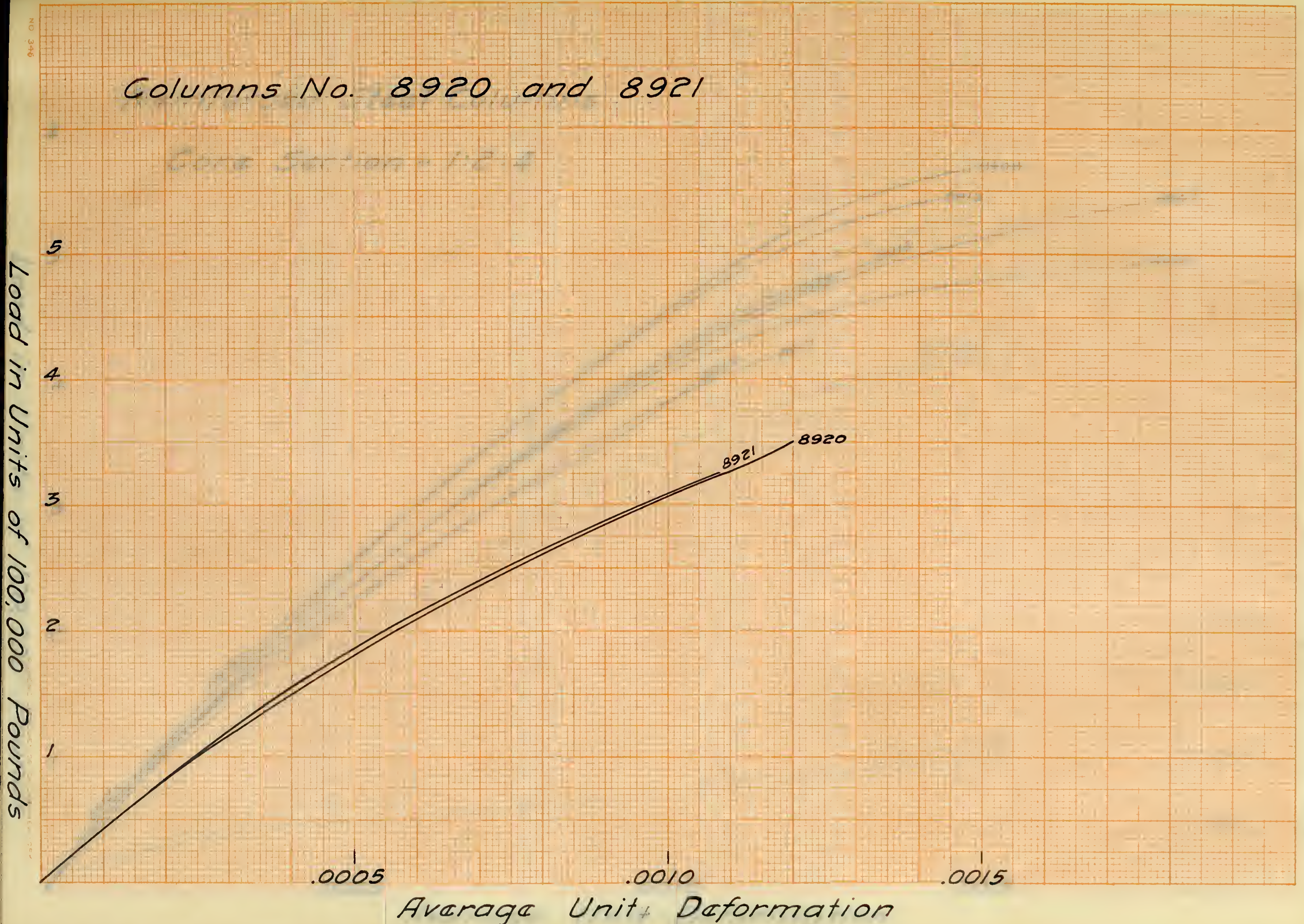
Column No. 8914 - Distribution of Deformation.
NE Flange
NW Flange
SW Flange
SE Flange
100 000
75000
50 000
25000
Load Considered on One Flange - lbs
Horizontal Scale 1 Inch = .0002 def.
Average Unit Deformation.

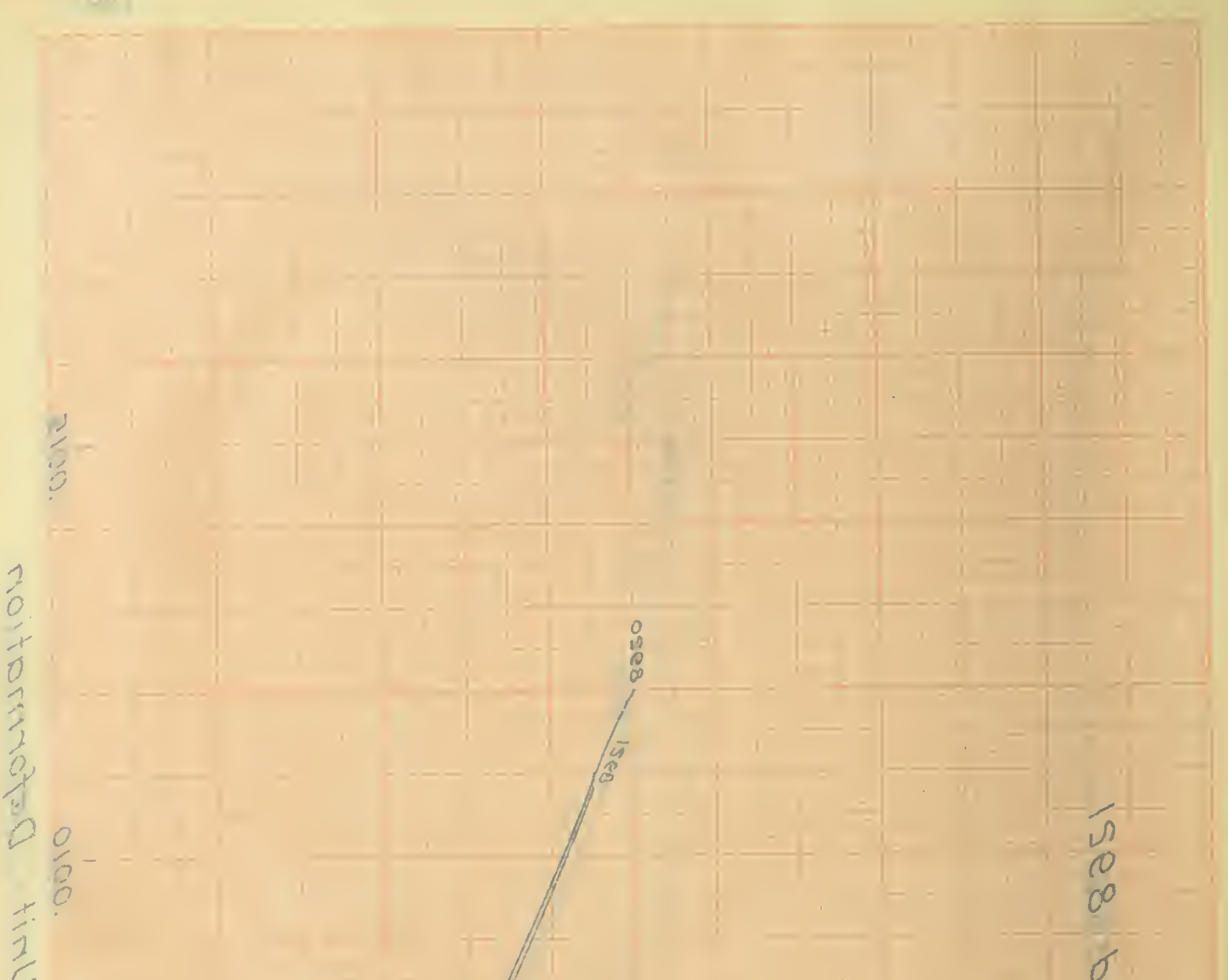


Column No. 8915 and 8916
5
4
3
2
1
8916
8915
.0005
.0010
.0015
Average Unit Deformation
Load in Units of 100,000 Pounds.

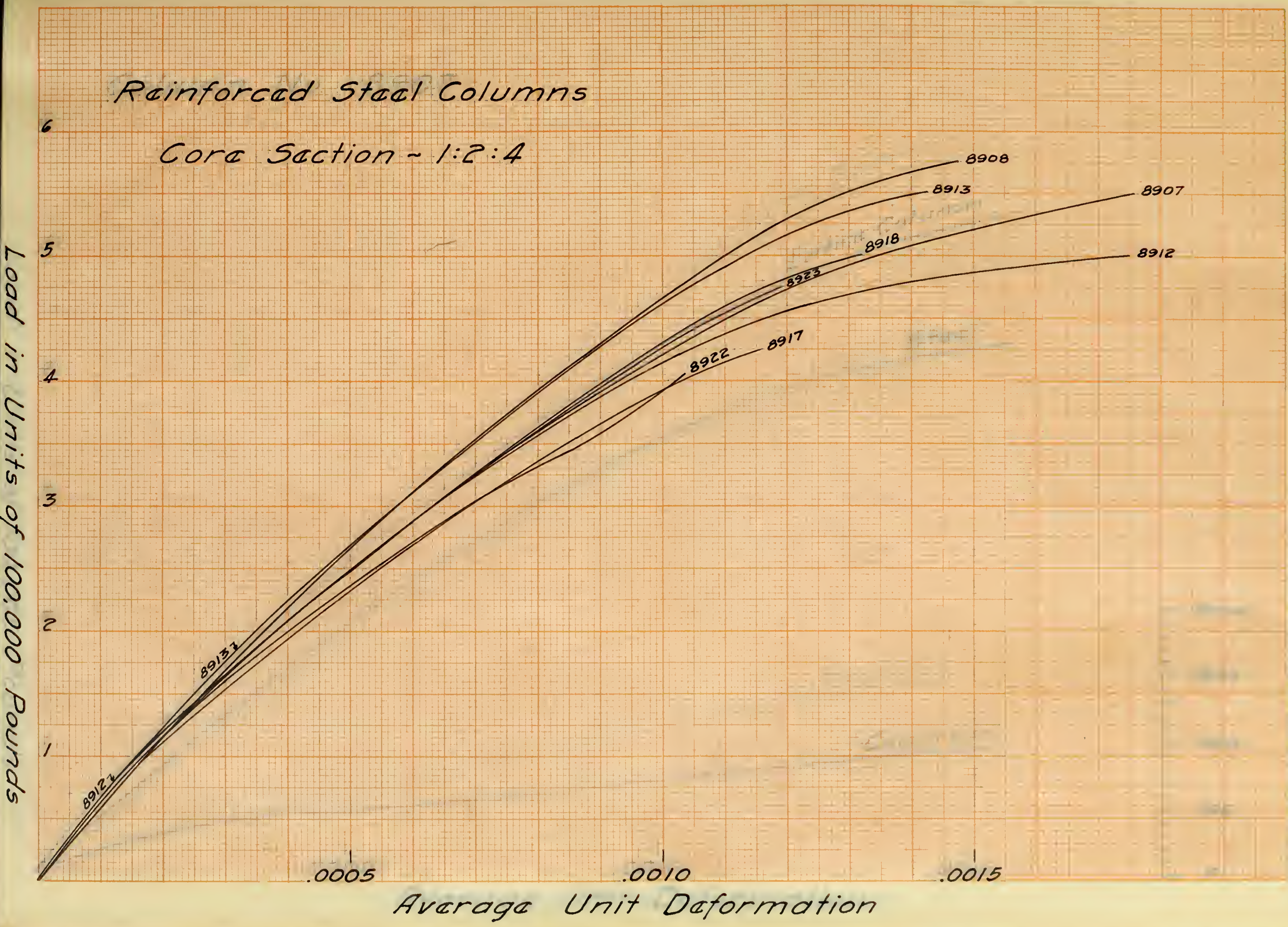


Column No. 8915 and 8916
Load in Units of 100,000 Pounds.
Average Unit Deformation
.0005
.0010
.0015
8915
8916
1
2
3
4
5

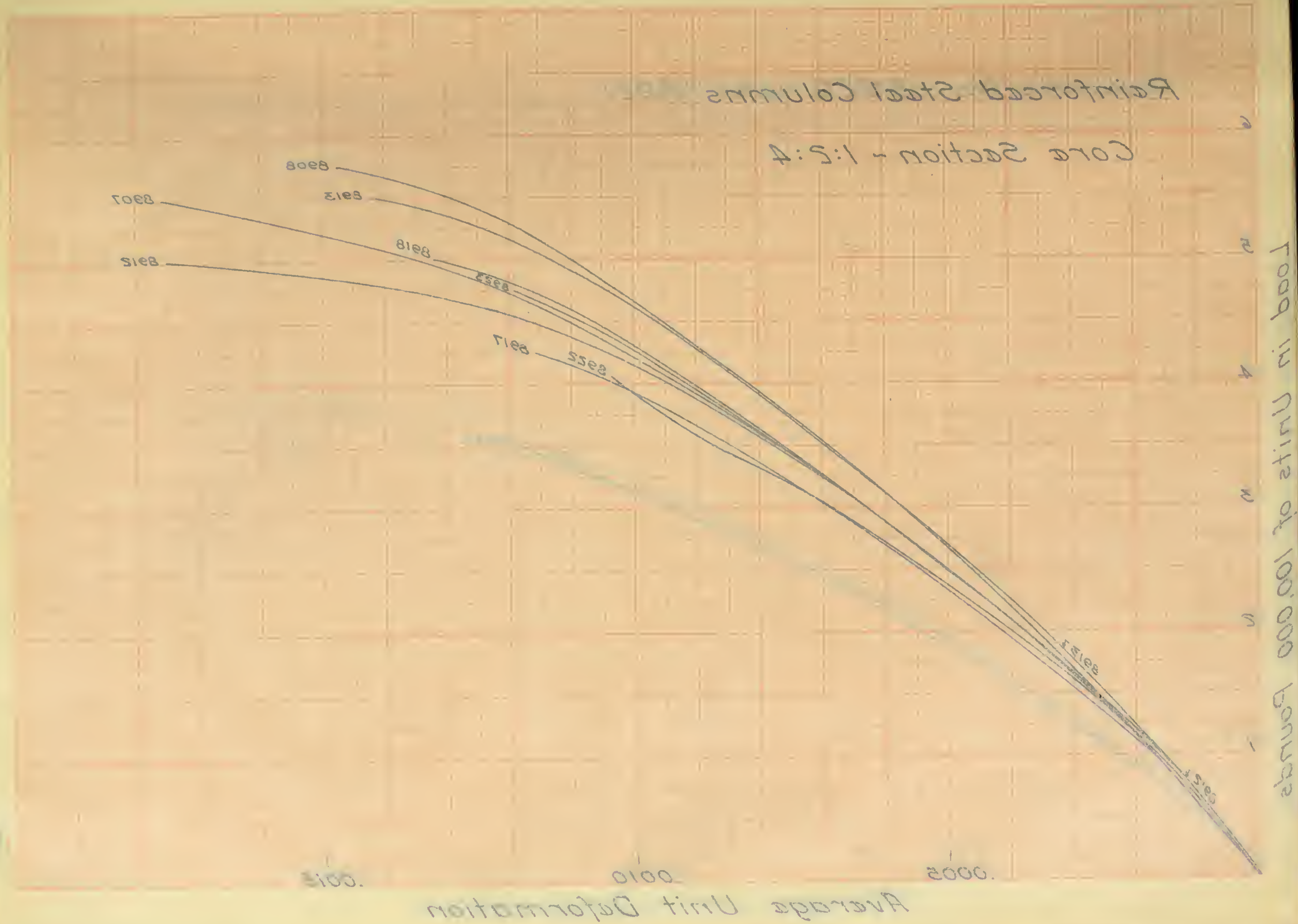

Columns No. 8920 and 8921
Load in Units of 100,000 Pounds
1
2
3
4
5
Average Unit Deformation
.0005
.0010
.0015
8920
8921

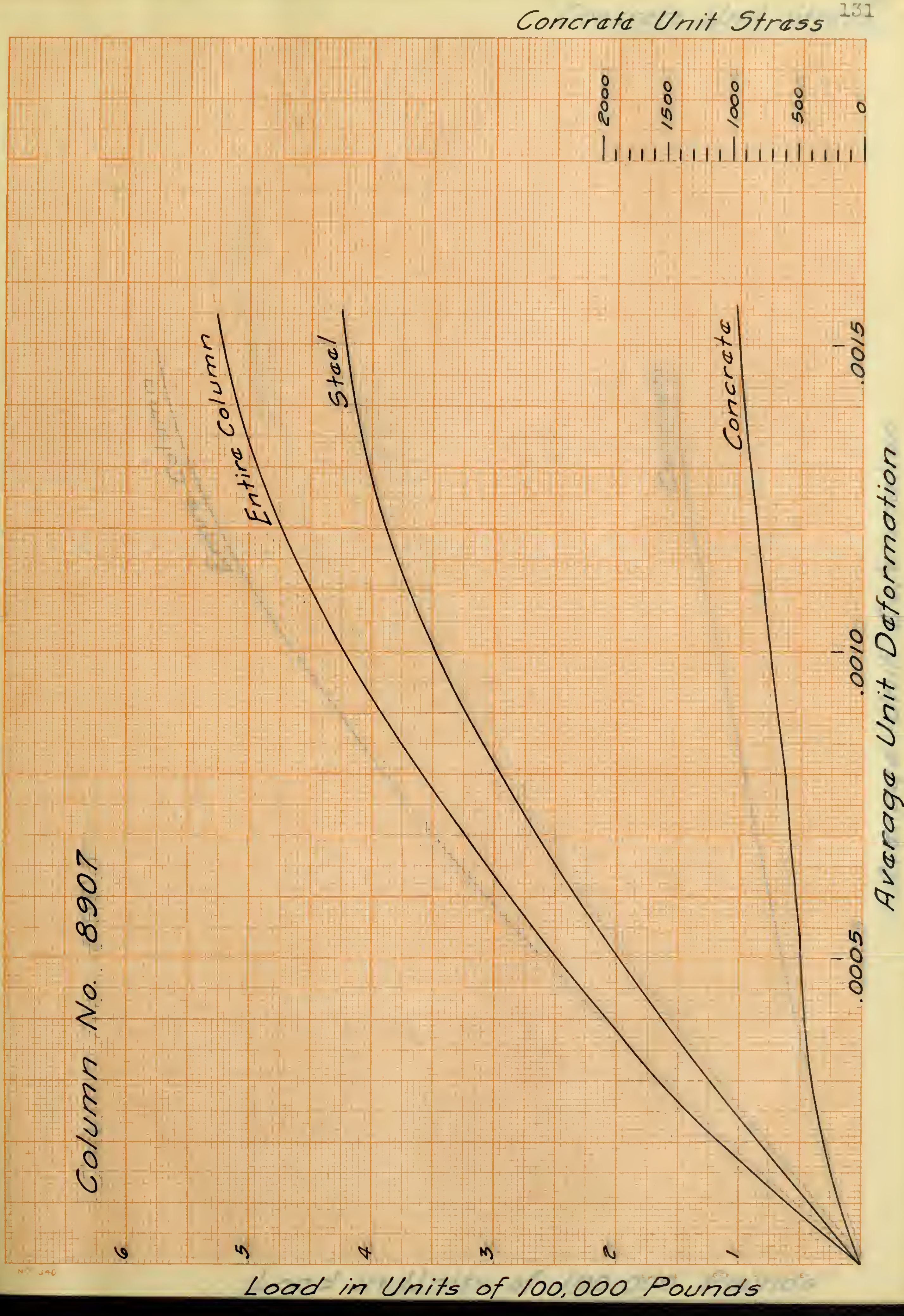


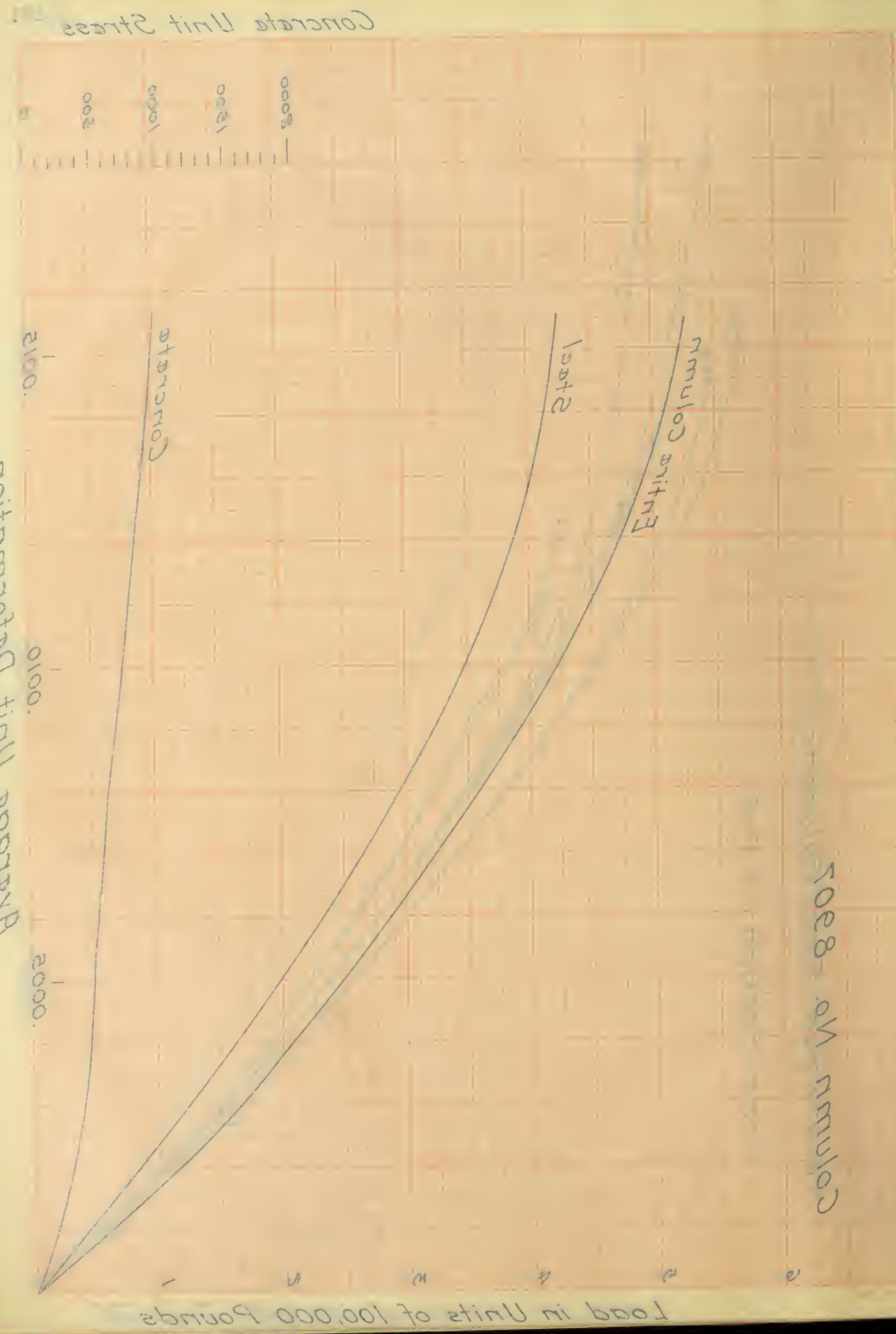
Columns No. 8920 and 8921
8920
8921
.0005
.0010
.0015
1
2
3
4
5
Average Unit Deformation
Load in Units of 100,000 Pounds



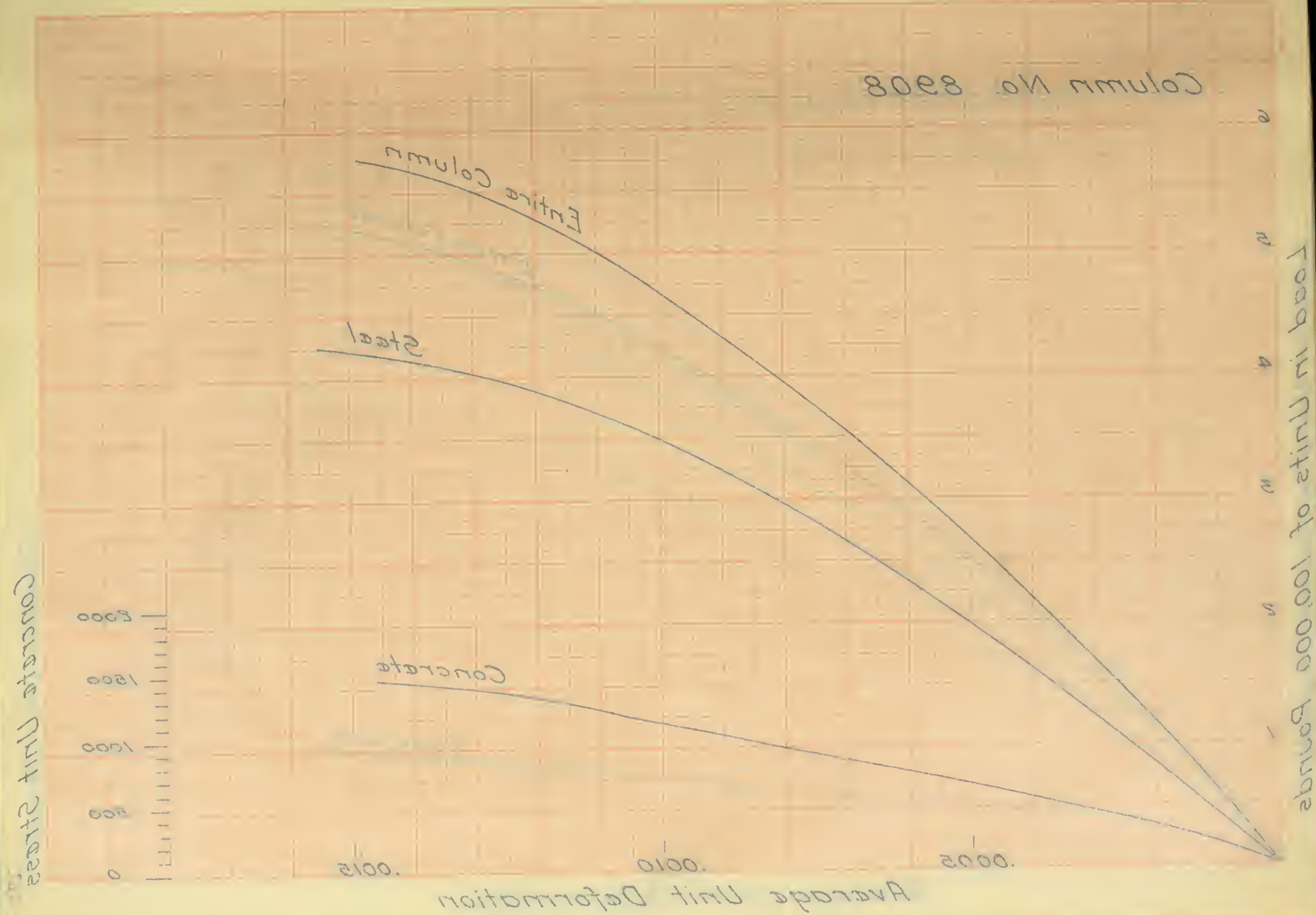
Reinforced Steel Columns
Core Section ~ 1:2:4
8908
8913
8907
8918
8912
8923
8917
8922
89131
89121
6
5
4
3
2
1
.0005
.0010
.0015
Load in Units of 100,000 Pounds
Average Unit Deformation


Reinforced Steel Columns
Core Section - 1:2:4
Load in Units of 100,000 Pounds
Average Unit Deformation
.0005
.0010
.0015
8908
8907
8913
8918
8912
8917
8922

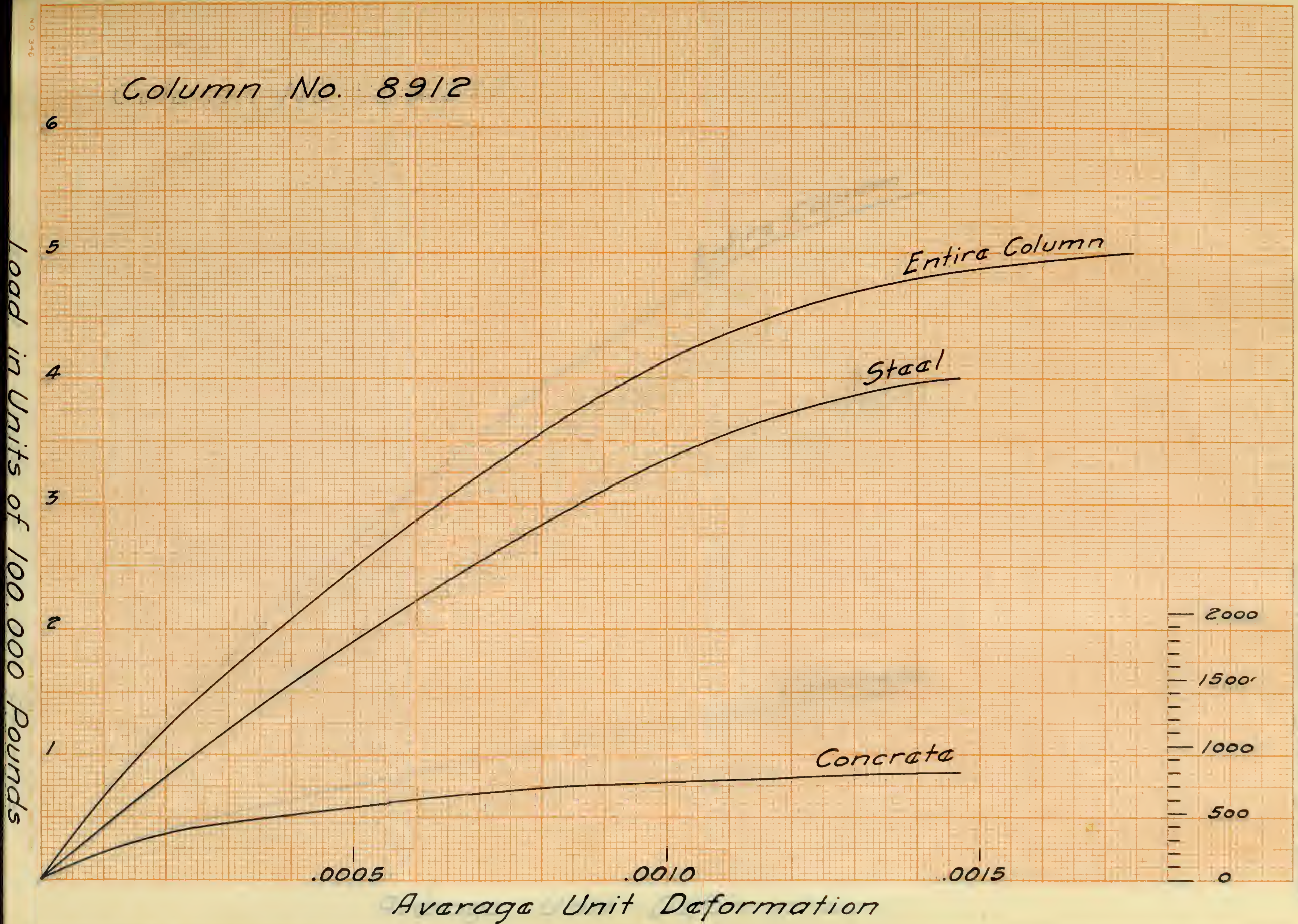

Concrete Unit Stress
0
500
1000
1500
2000
Column No. 8907
Entire Column
Steel
Concrete
.0005
.0010
.0015
Average Unit Deformation
1
2
3
4
5
6
Load in Units of 100,000 Pounds

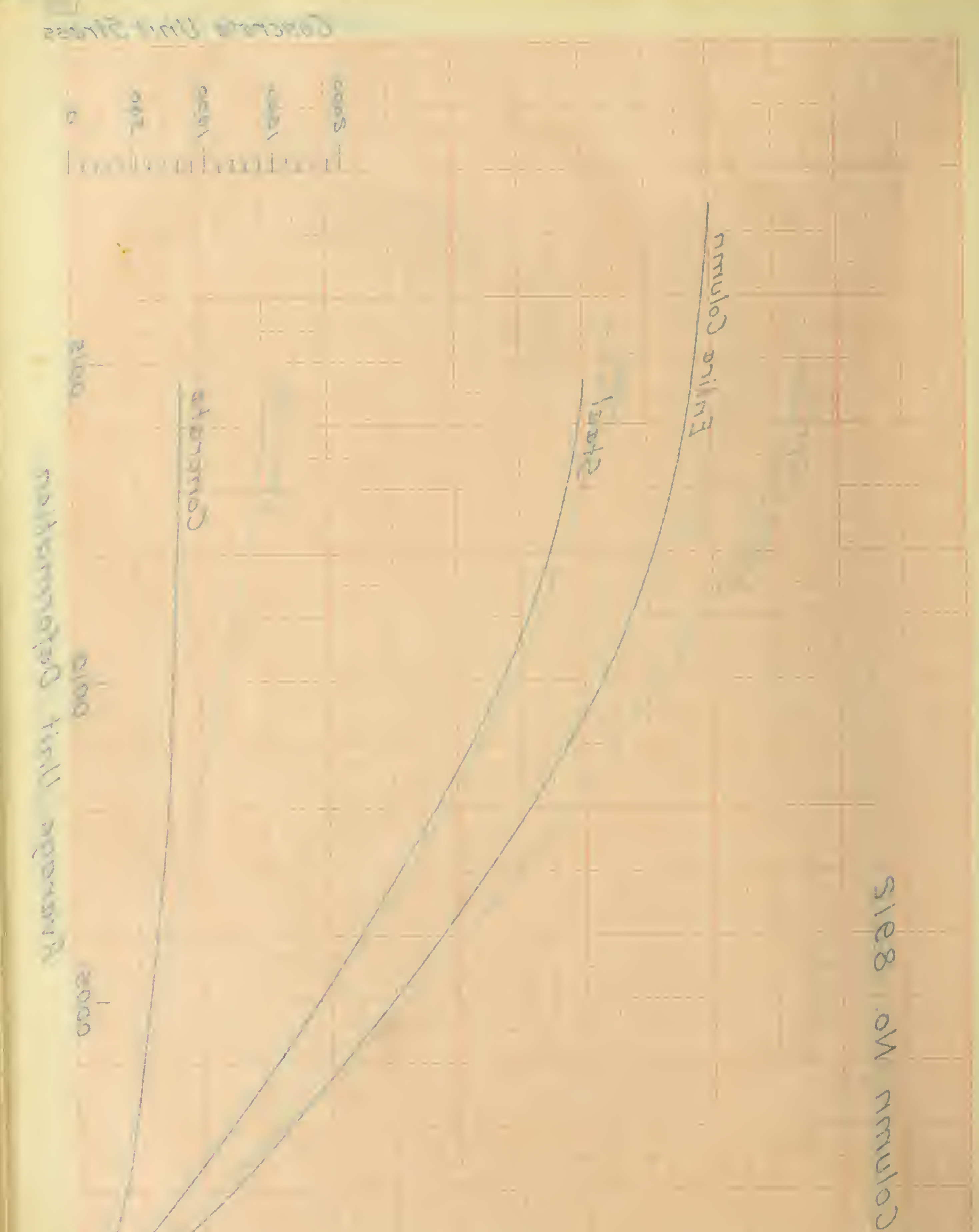

Column No. 8907
Concrete Unit Stress
500
1000
1500
2000
Average Unit Deformation
.0005
.0010
.0015
Load in Units of 100,000 Pounds
Concrete
Steel
Entire Column


Concrete Unit Stress
2000
1500
1000
500
0
Column No. 8908
Entire Column
Steel
Concrete
.0015
.0010
.0005
Average Unit Deformation
6
5
4
3
2
1
Load in Units of 100,000 Pounds
NO. 346

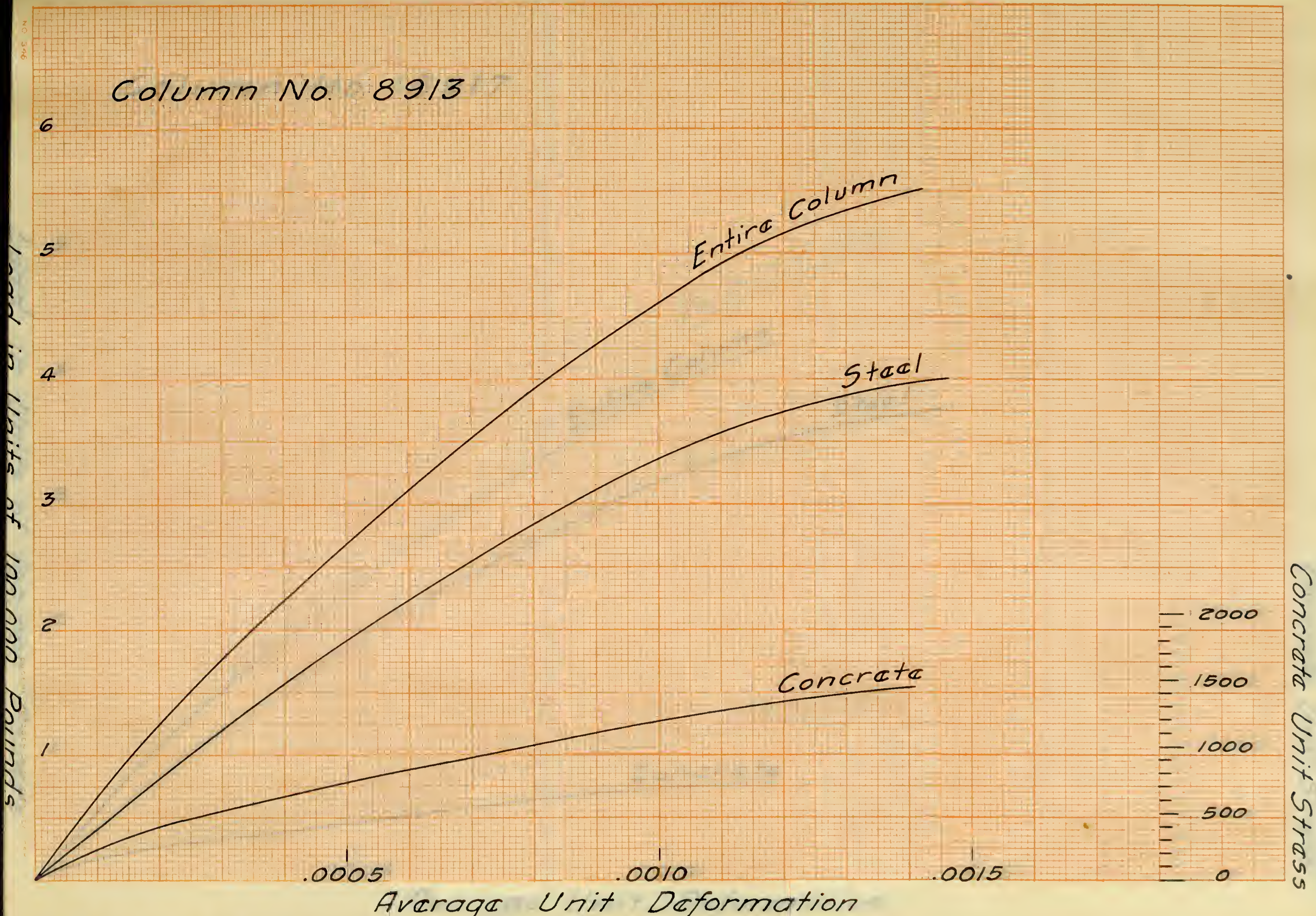


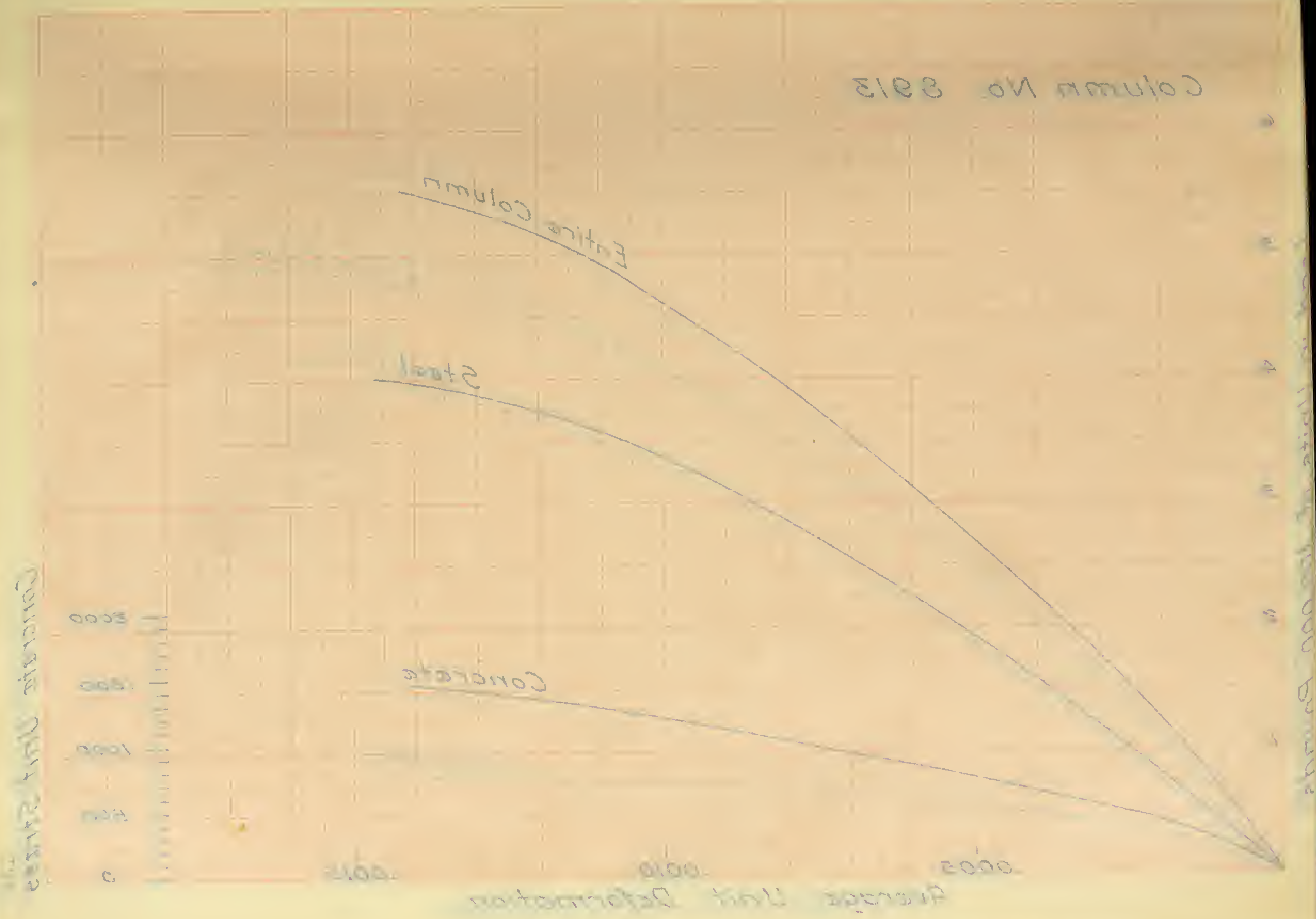
Column No. 8908
Entire Column
Steel
Concrete
Concrete Unit Stress
0
500
1000
1500
2000
.0005
.0010
.0015
Average Unit Deformation
1
2
3
4
5
6
Load in Units of 100,000 Pounds

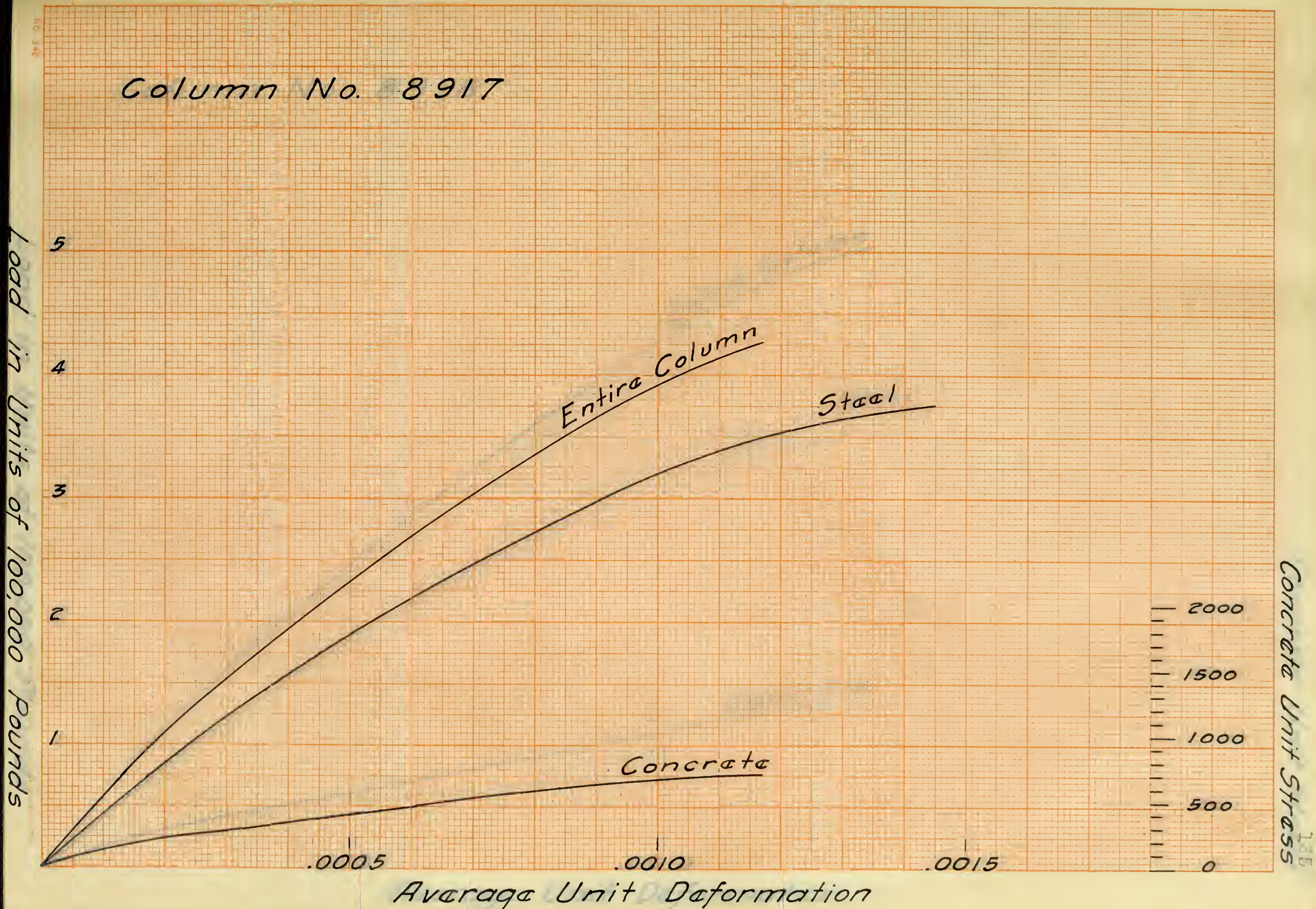

Column No. 8912
Entire Column
Steel
Concrete
6
5
4
3
2
1
.0005
.0010
.0015
2000
1500
1000
500
0
Load in Units of 100.000 Pounds
Average Unit Deformation
Concrete Unit Stress



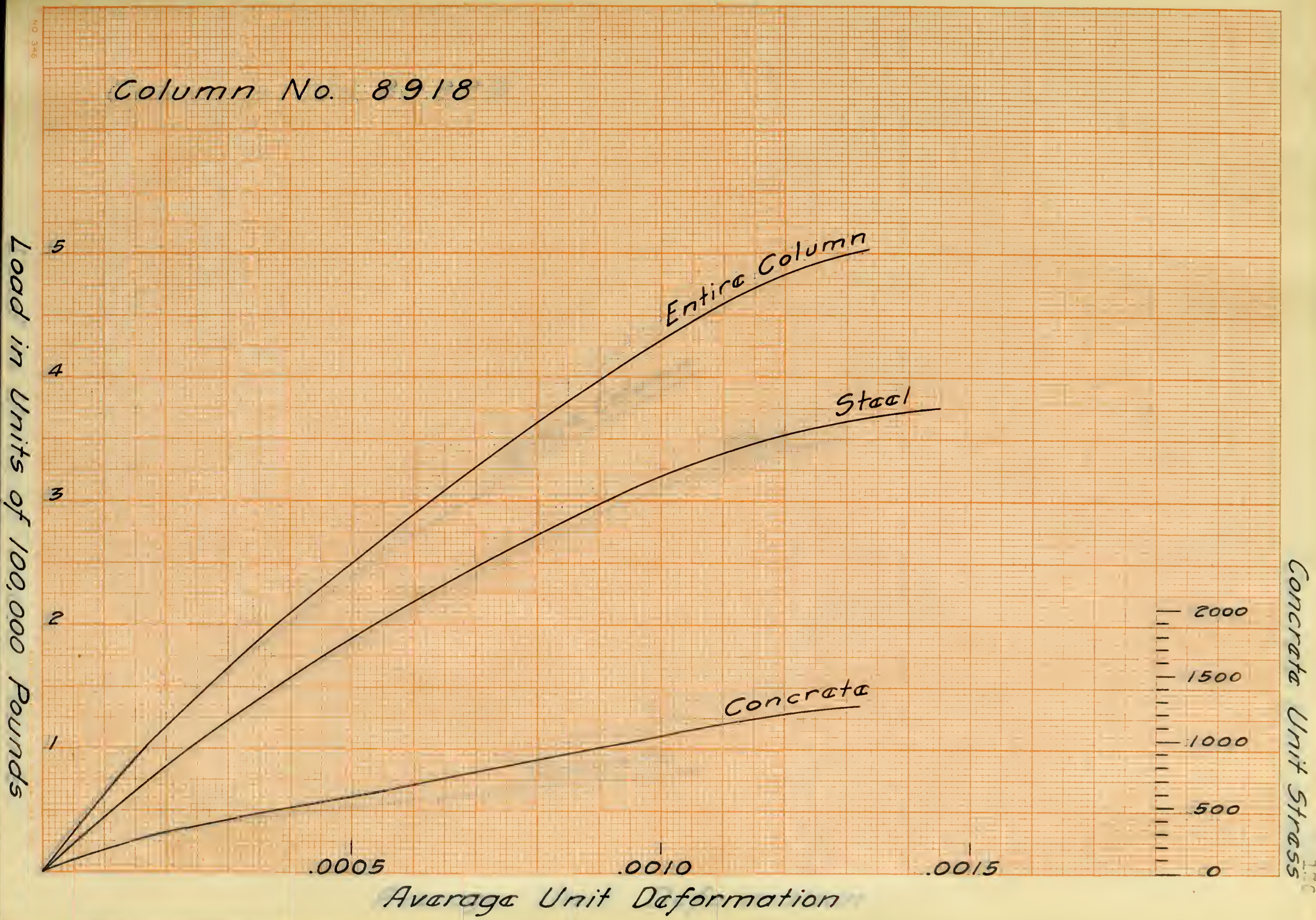
Column No. 8912
Entire Column
Steel
Concrete
Load in Units of 100 000 Pounds
Average Unit Deformation
Concrete Unit Stress


Column No. 8913
Concrete Unit Stress
2000
1500
1000
500
0
Entire Column
Steel
Concrete
Average Unit Deformation
.0005
.0010
.0015
Load in Units of 100,000 Pounds
1
2
3
4
5
6


Column No.
Entire Column
Steel
Concrete
Concrete Unit Stress
0
500
1000
1500
2000
.0005
.0010
.0015
Average Unit Deformation
Load in Units of 100,000 Pounds

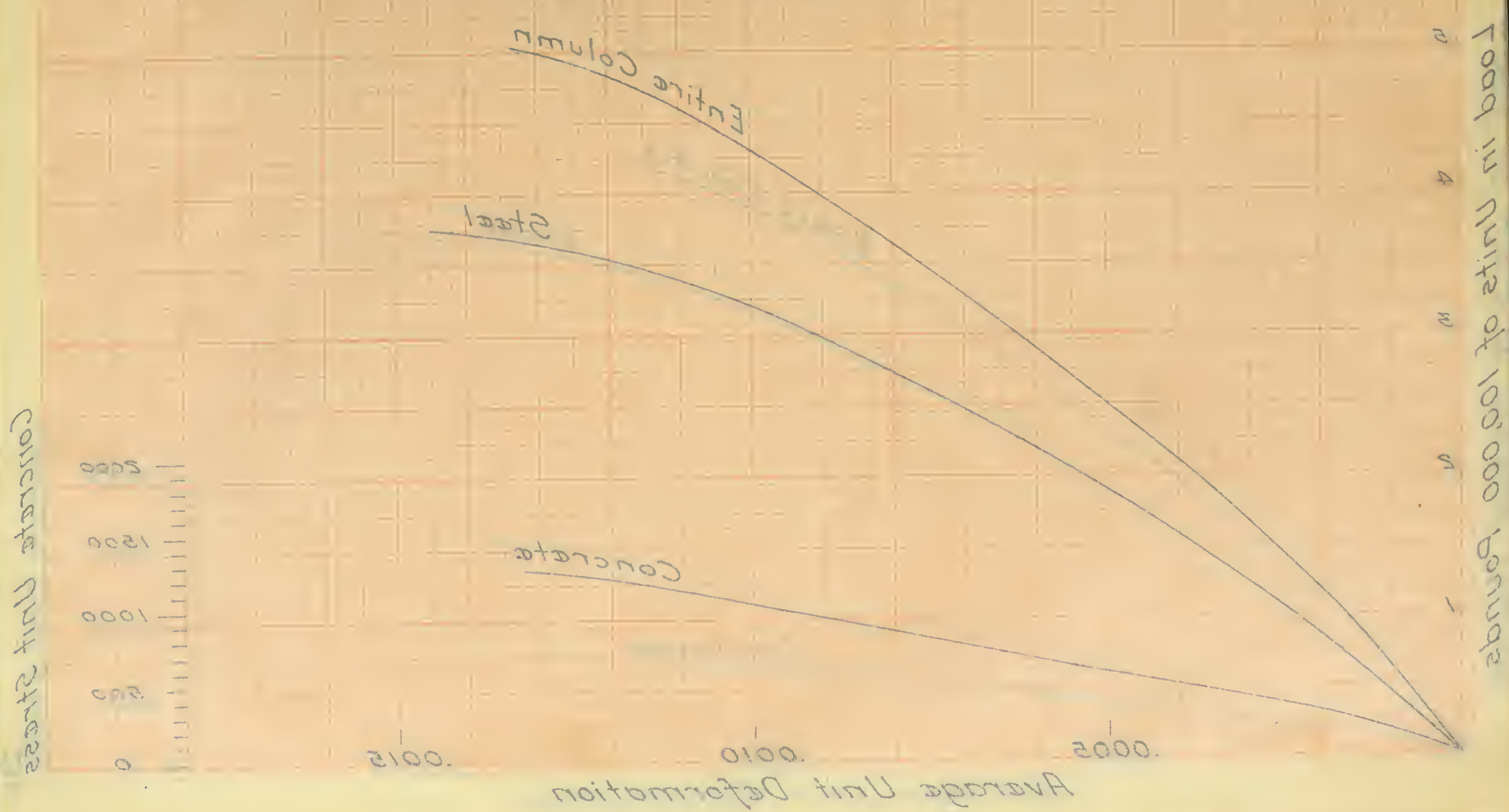

Column No. 8917
Entire Column
Steel
Concrete
5
4
3
2
1
Load in Units of 100,000 Pounds
.0005
.0010
.0015
Average Unit Deformation
2000
1500
1000
500
0
Concrete Unit Stress

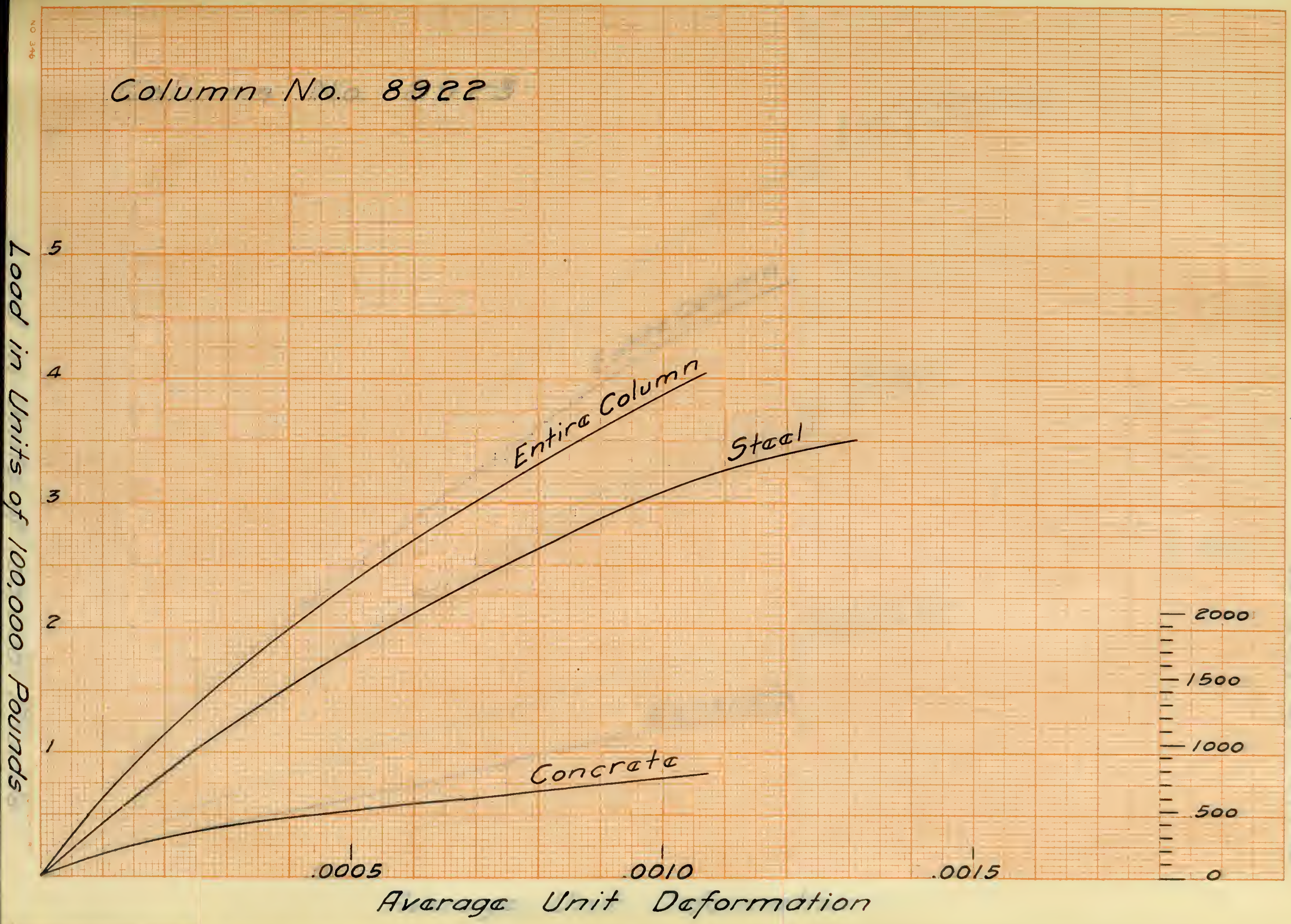

Column No. 8917
Steel
Entire Column
Concrete
Concrete Unit Stress
0
500
1000
1500
2000
.0015
.0010
.0005
Average Unit Deformation
1
2
3
4
5
Load in Units of 100,000 Pounds



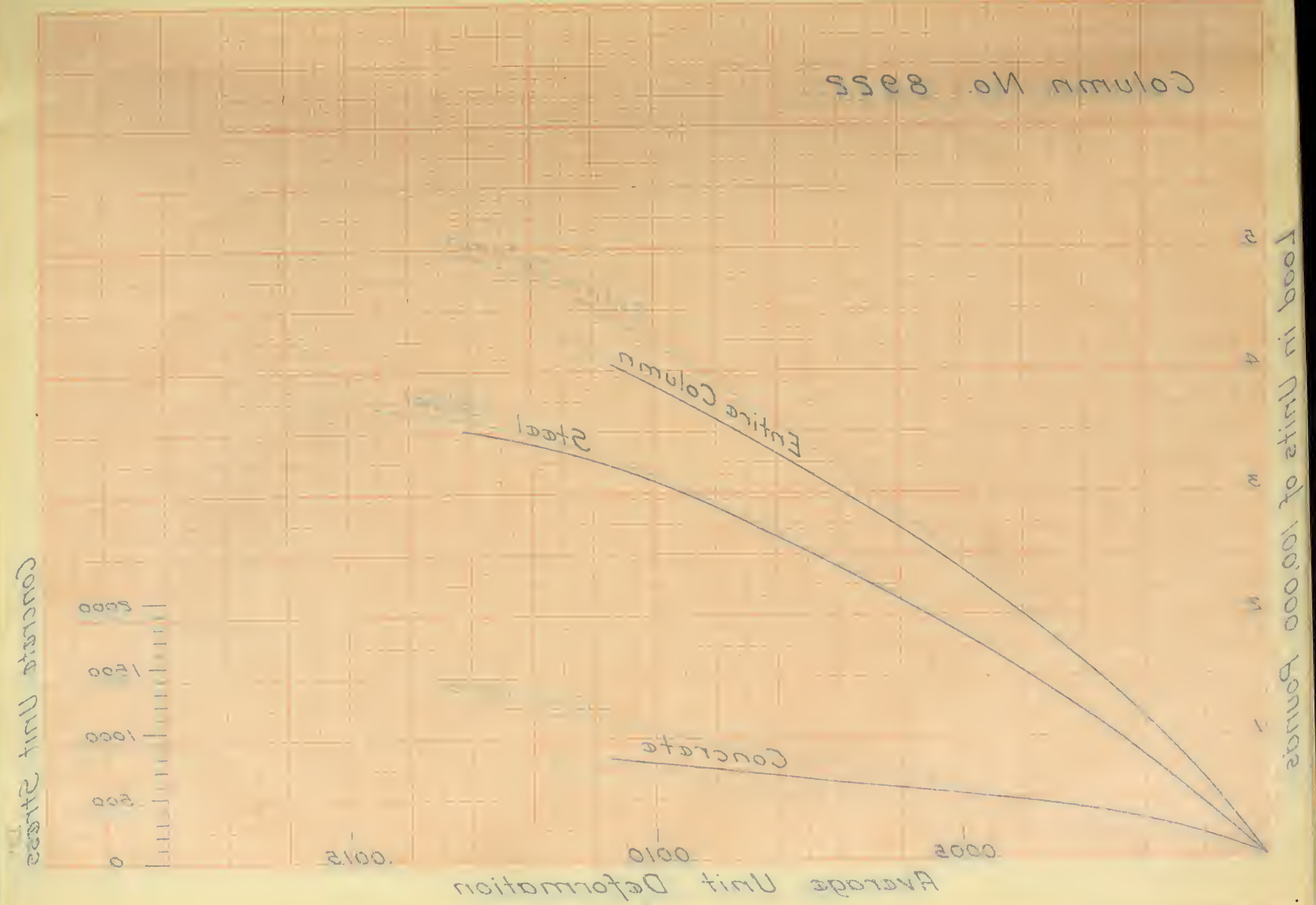
Column No. 8918
Entire Column
Steel
Concrete
5
4
3
2
1
.0005
.0010
.0015
2000
1500
1000
500
0
Load in Units of 100,000 Pounds
Average Unit Deformation
Concrete Unit Stress

Column No. 8918

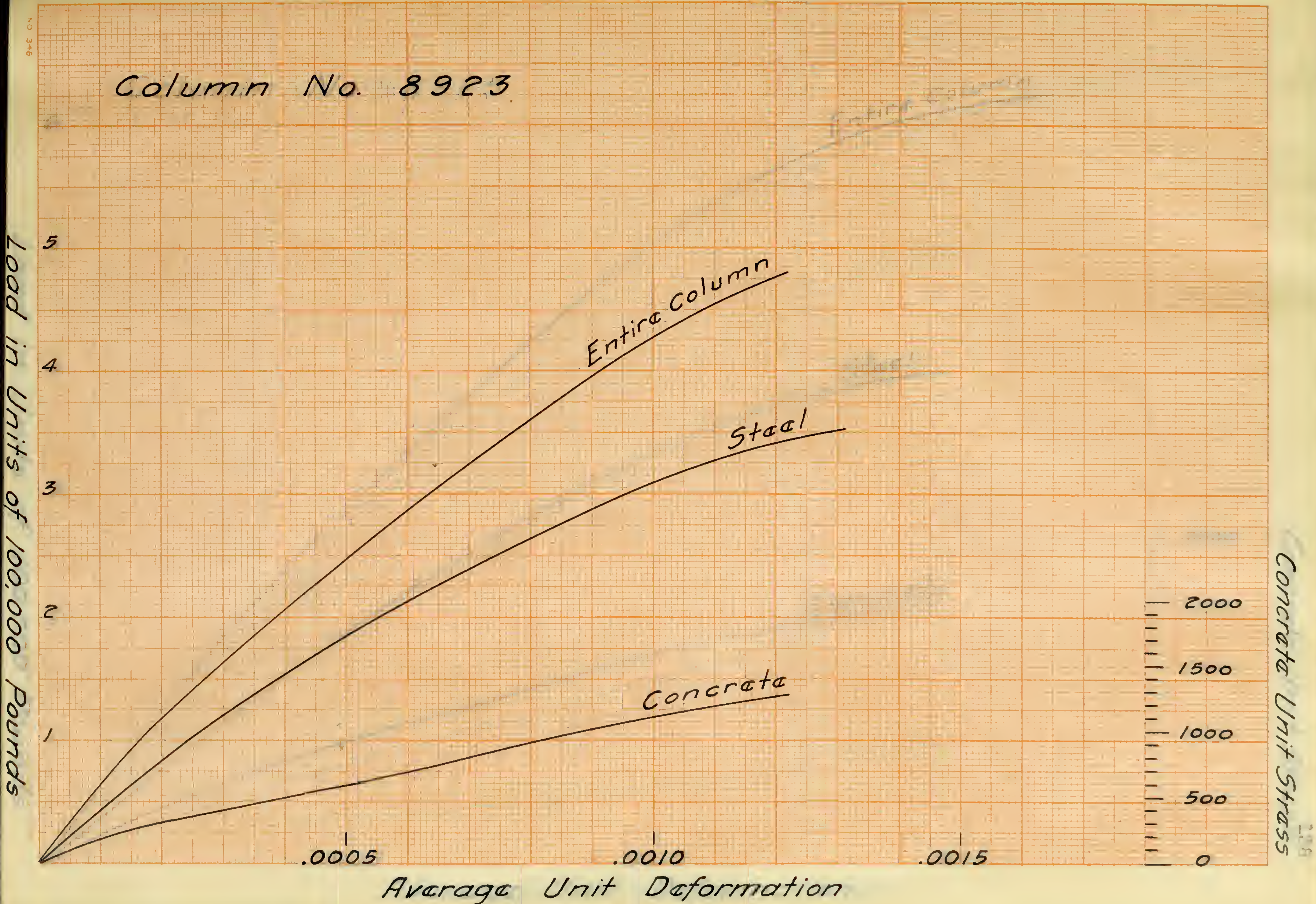




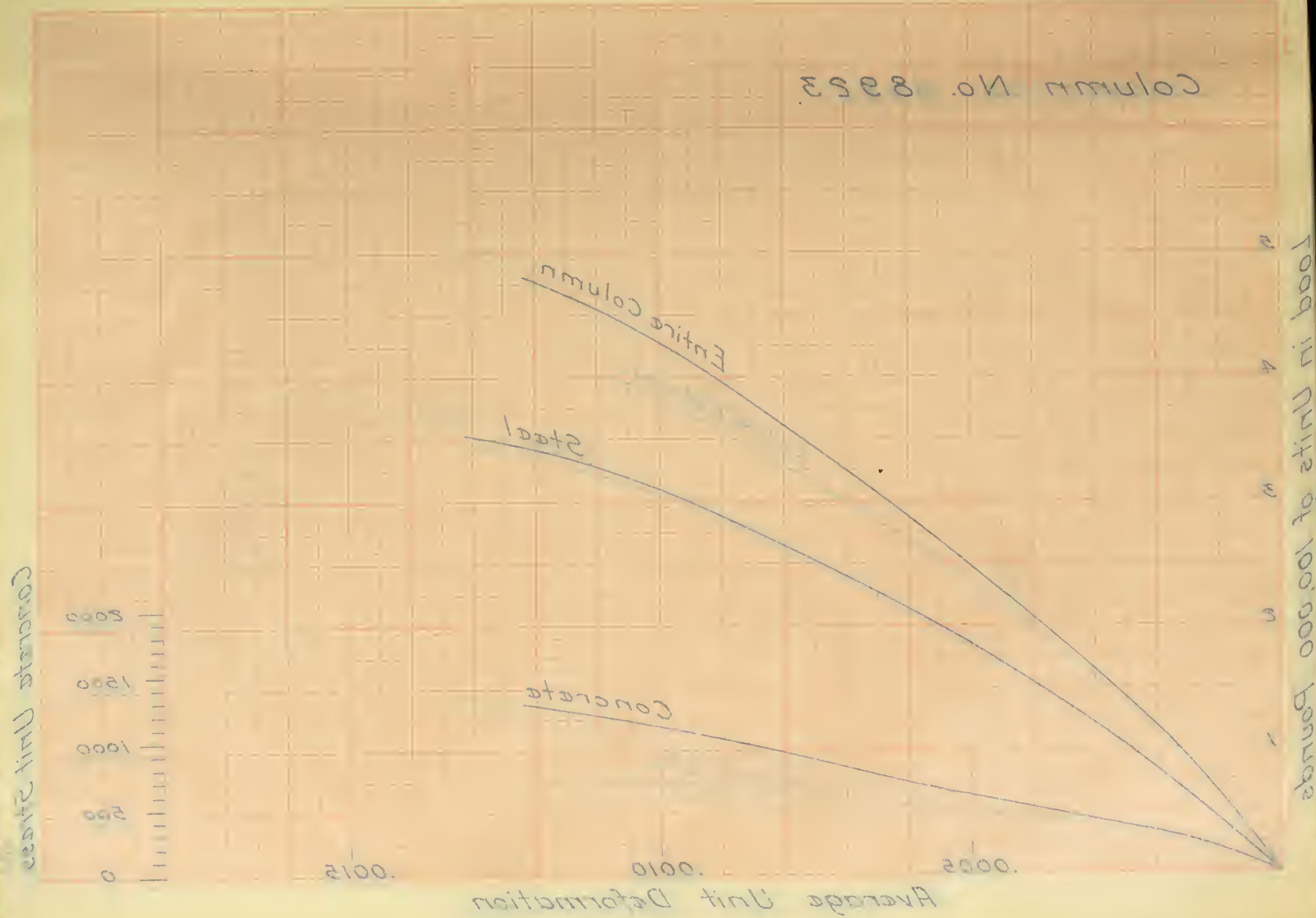
Column No. 8922
Load in Units of 100,000 Pounds
5
4
3
2
1
Entire Column
Steel
Concrete
.0005
.0010
.0015
Average Unit Deformation
Concrete Unit Stress
2000
1500
1000
500
0

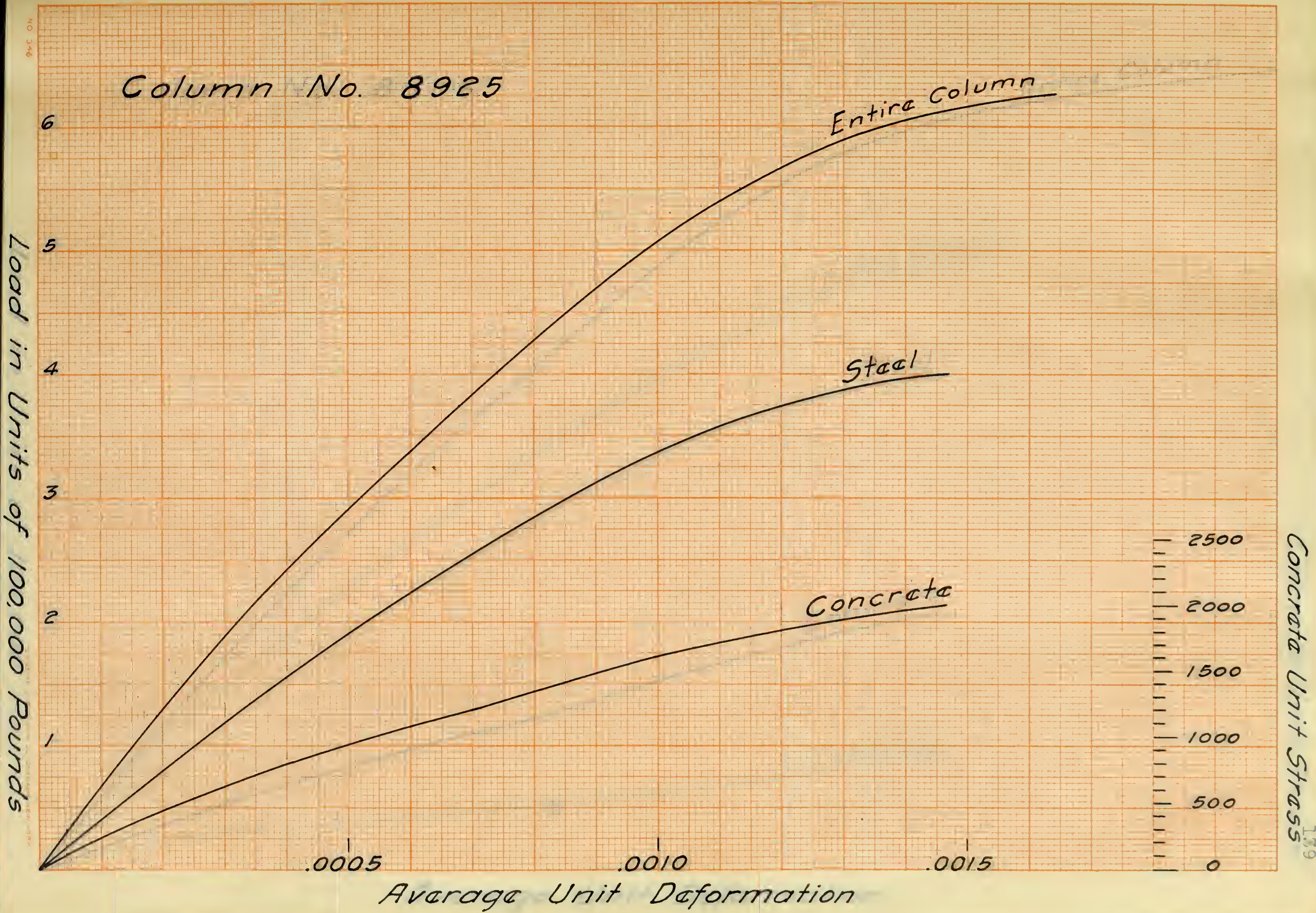


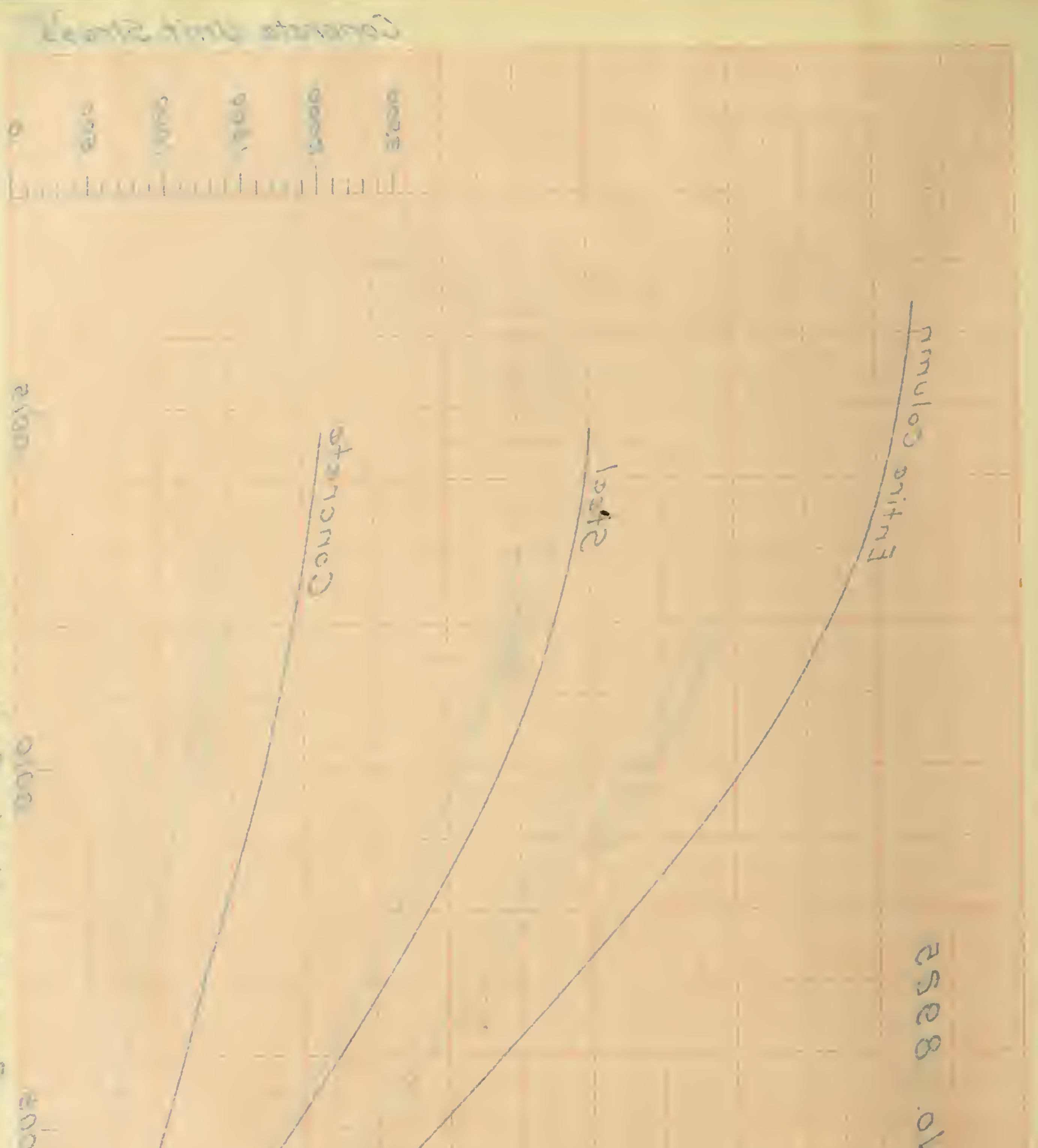
Column No. 8922
Concrete Unit Stress
0
500
1000
1500
2000
Average Unit Deformation
.0005
.0010
.0015
Concrete
Steel
Entire Column
1
2
3
4
5
Load in Units of 100,000 Pounds

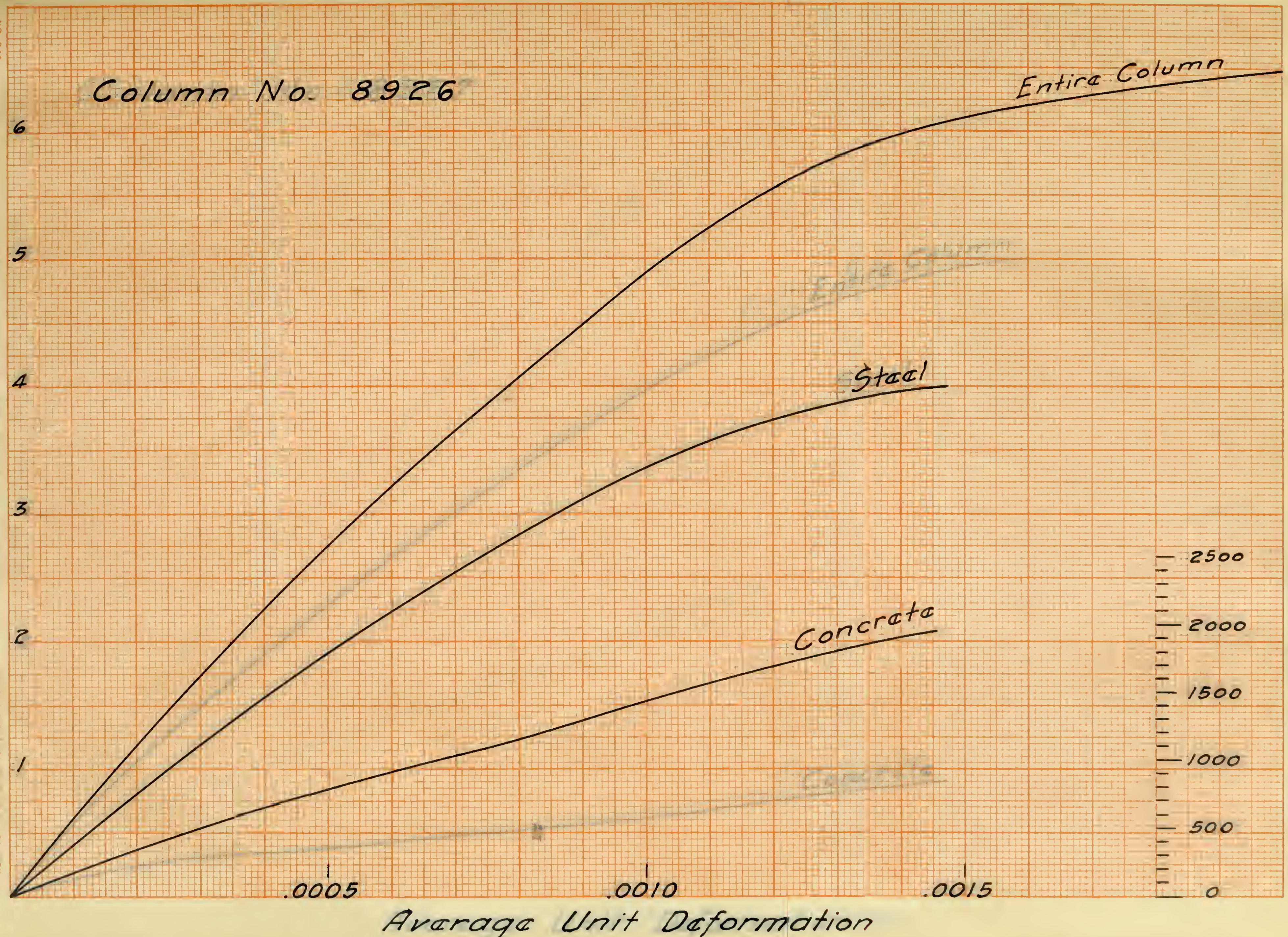


Column No. 8923
Entire Column
Steel
Concrete
5
4
3
2
1
.0005
.0010
.0015
2000
1500
1000
500
0
Average Unit Deformation
Load in Units of 100,000 Pounds
Concrete Unit Stress
NO 346

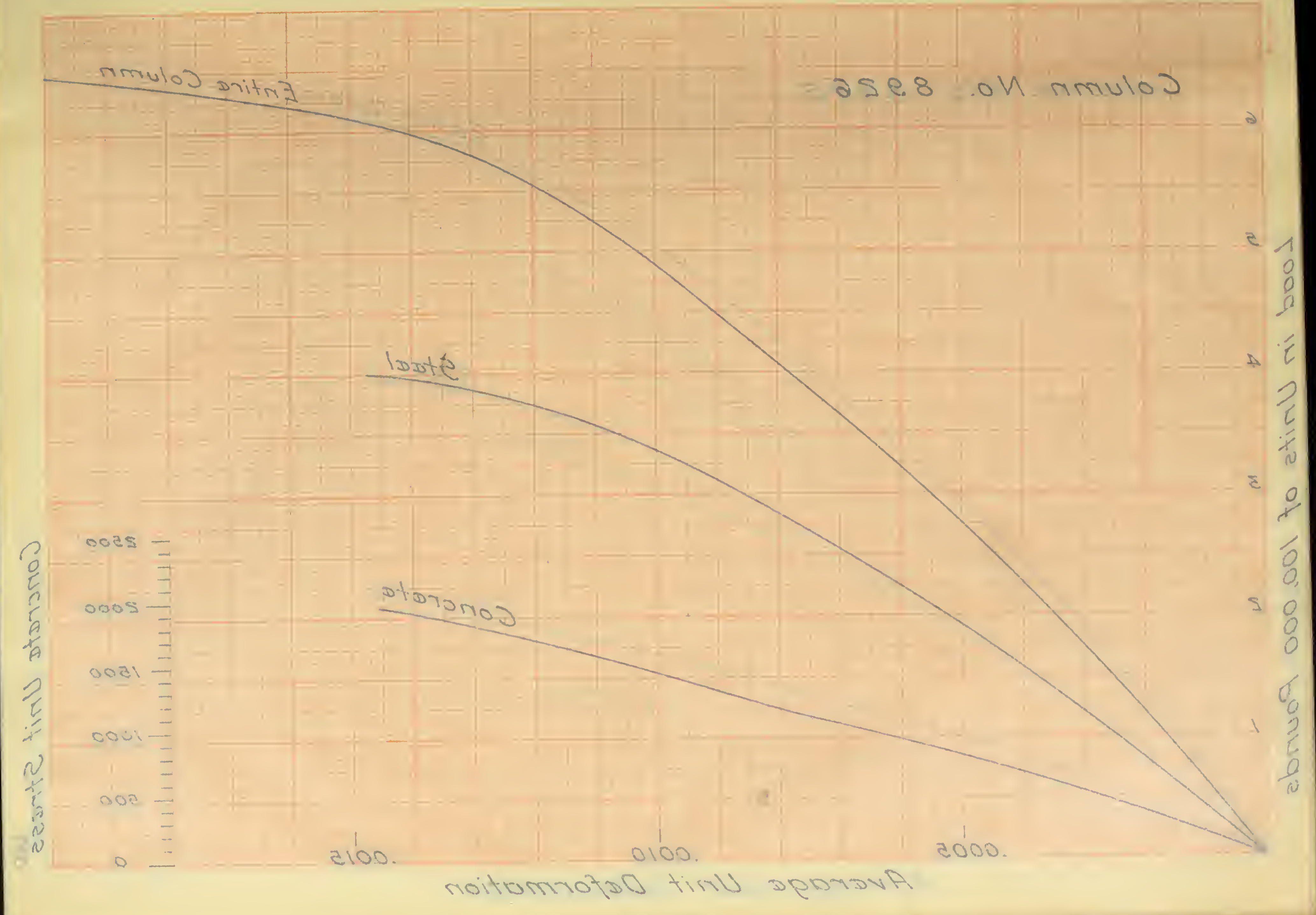

Column No. 8923
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
1
2
3
4
5
Concrete Unit Stress
0
500
1000
1500
2000
.0005
.0010
.0015
Average Unit Deformation

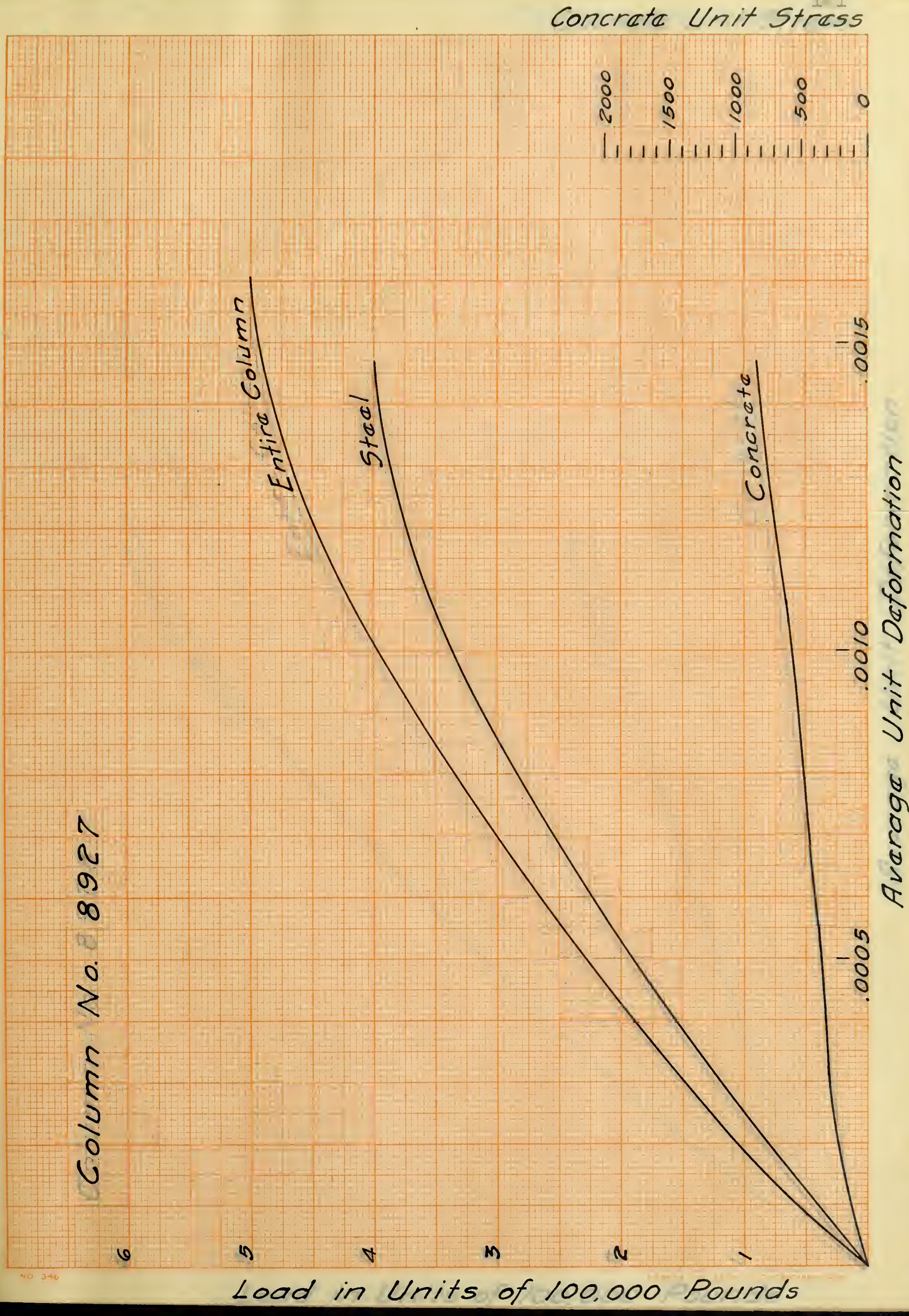

Column No. 8925
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
Average Unit Deformation
Concrete Unit Stress
.0005
.0010
.0015
1
2
3
4
5
6
0
500
1000
1500
2000
2500

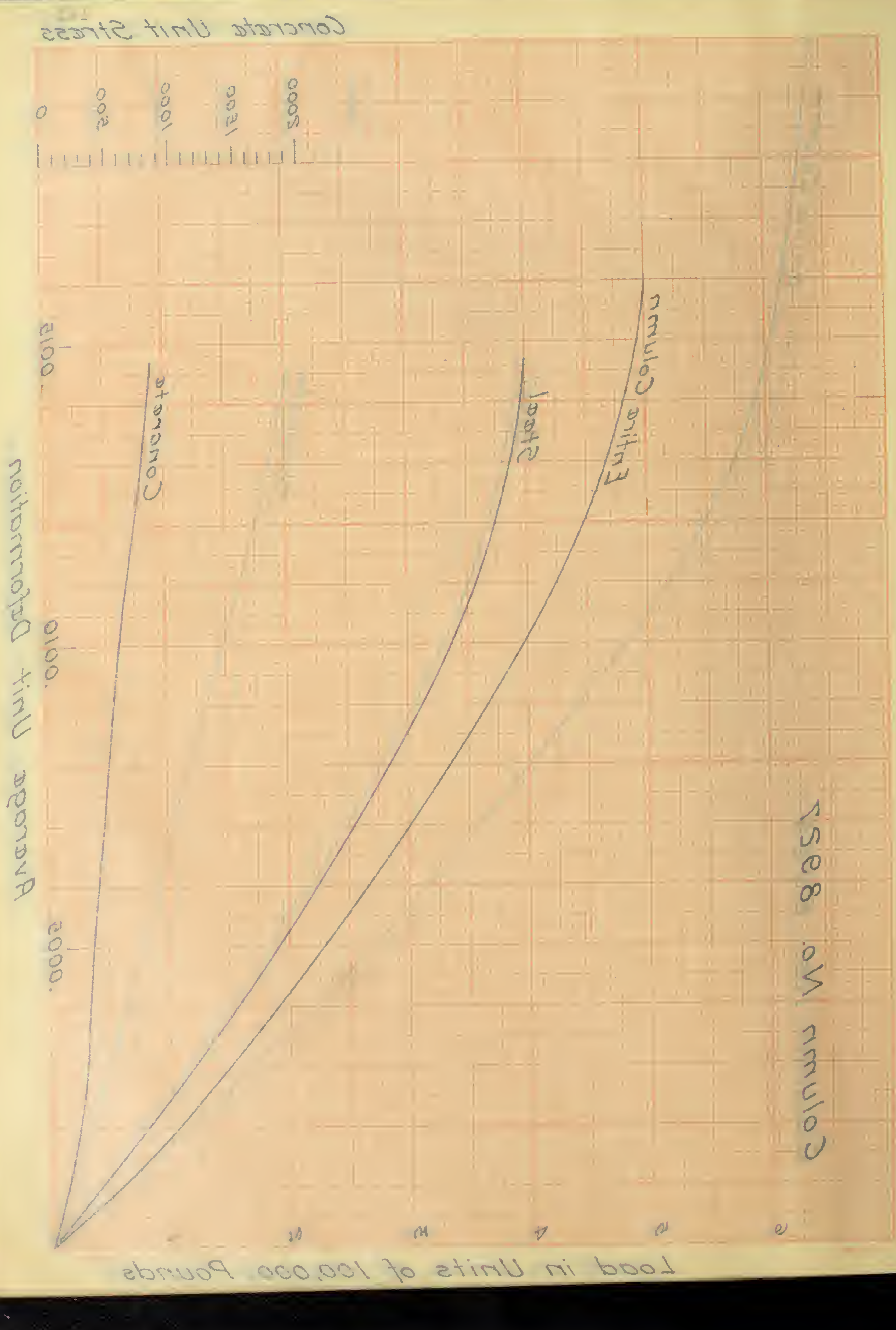

Column No. 8925
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
Average Unit Deformation
Concrete Unit Stress



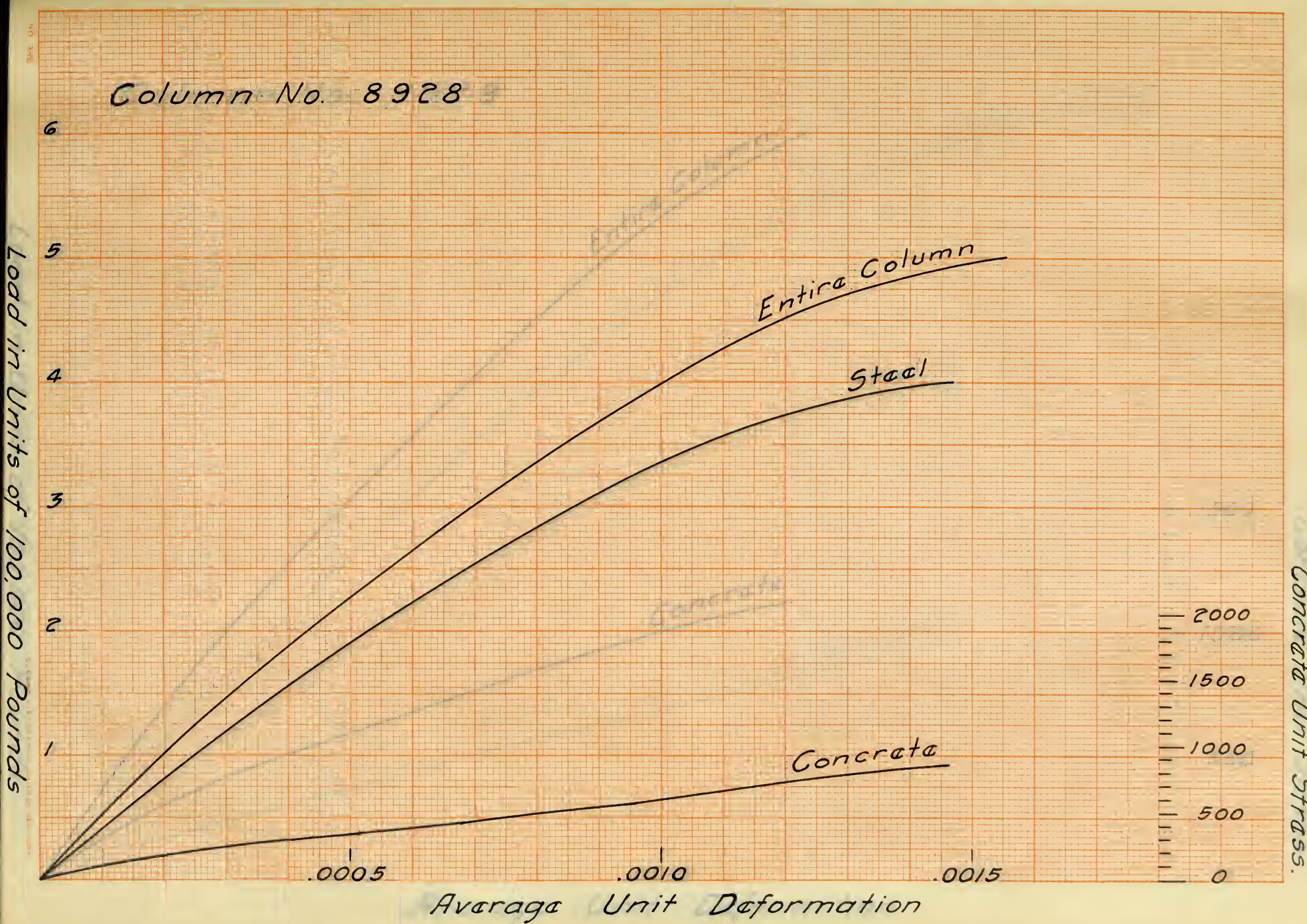
Column No. 8926
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
1
2
3
4
5
6
.0005
.0010
.0015
Average Unit Deformation
Concrete Unit Stress
0
500
1000
1500
2000
2500


Column No. 8926
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
Average Unit Deformation
.0005
.0010
.0015
Concrete Unit Stress
0
500
1000
1500
2000
2500
1
2
3
4
5
6

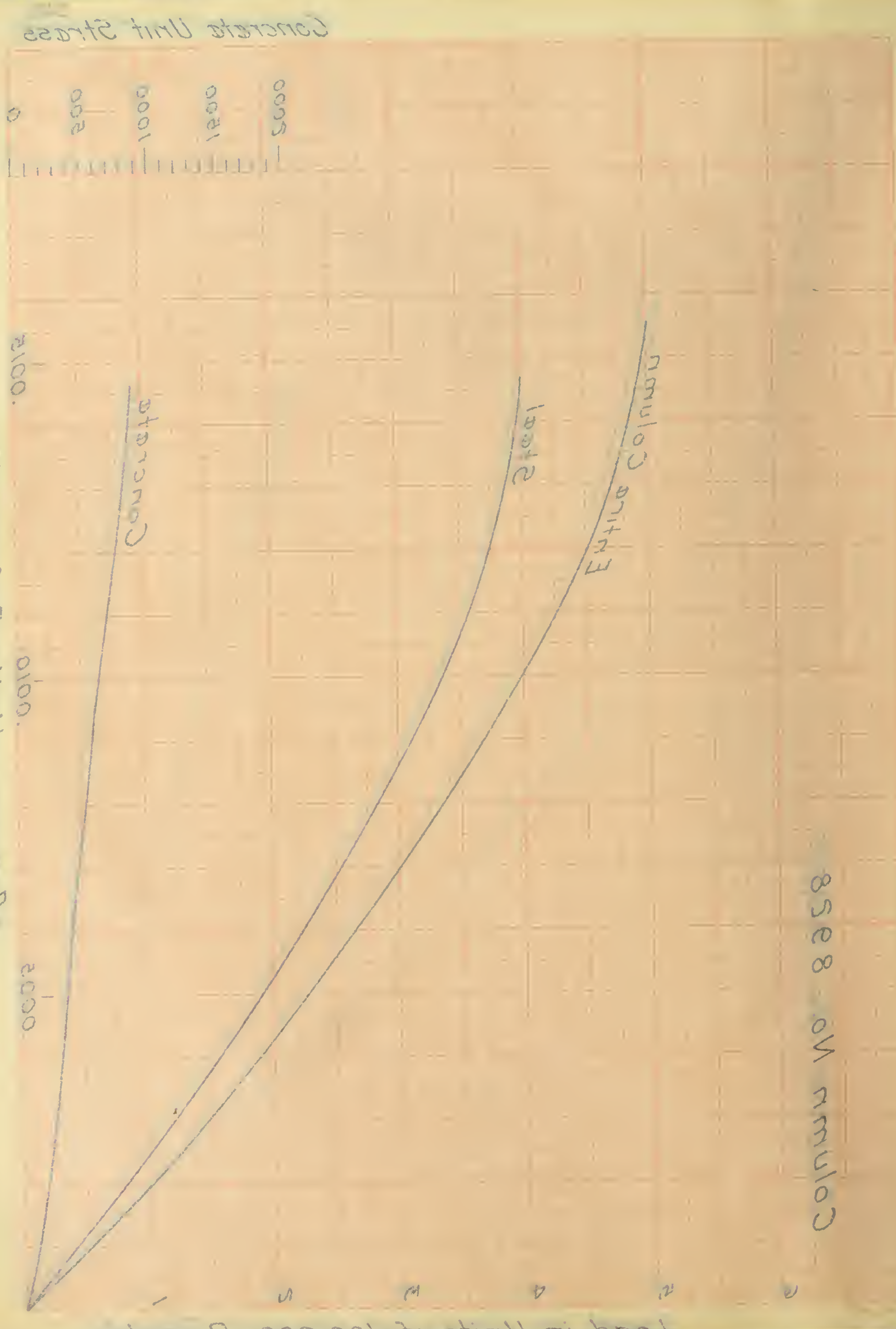

Concrete Unit Stress
2000
1500
1000
500
0
Column No. 8927
Entire Column
Steel
Concrete
.0015
.0010
.0005
Average Unit Deformation
6
5
4
3
2
1
Load in Units of 100,000 Pounds



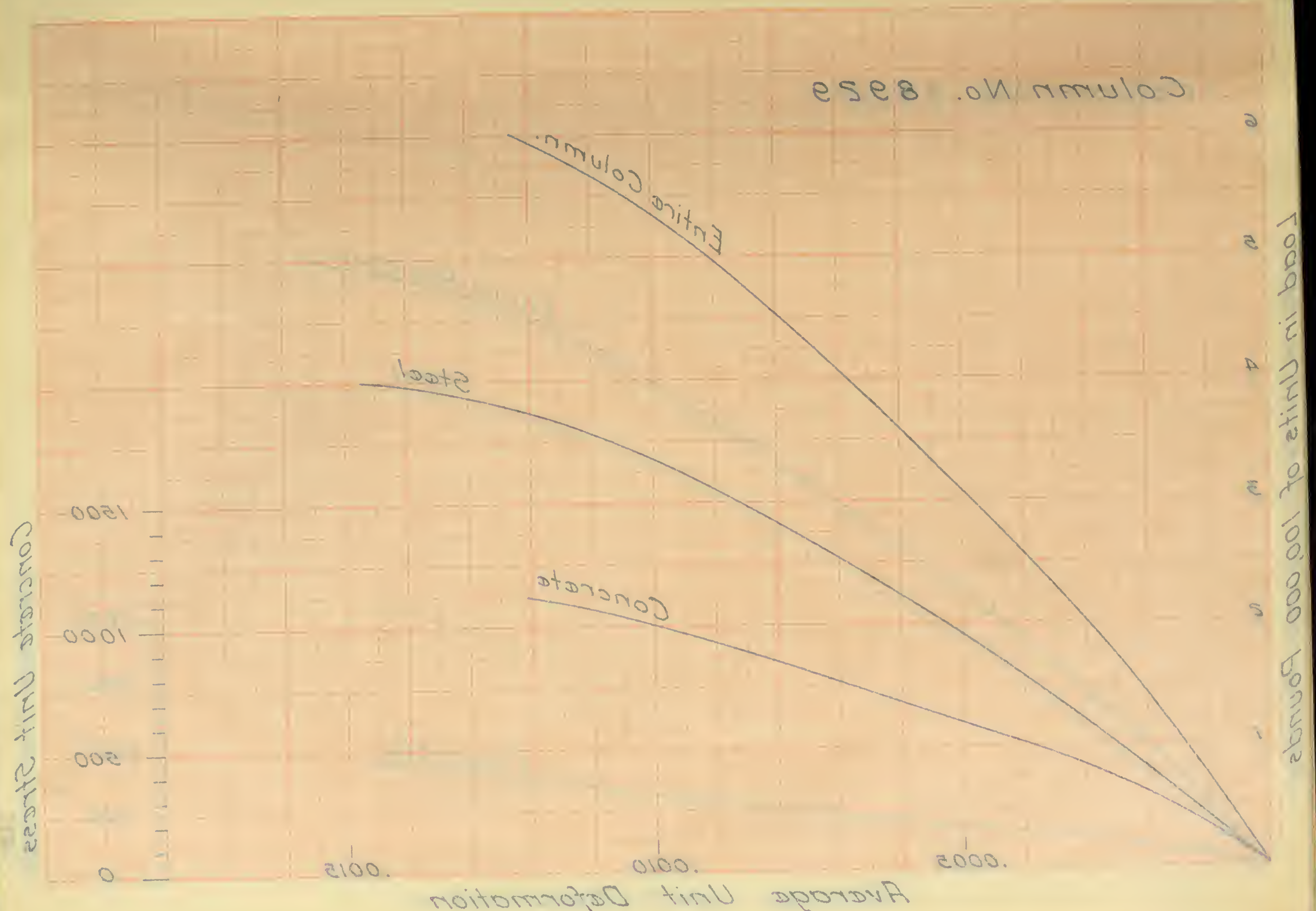
Concrete Unit Stress
0
500
1000
1500
2000
Concrete
Steel
Entire Column
Column No. 8927
.0005
.0010
.0015
Average Unit Deformation
Load in Units of 100,000 Pounds

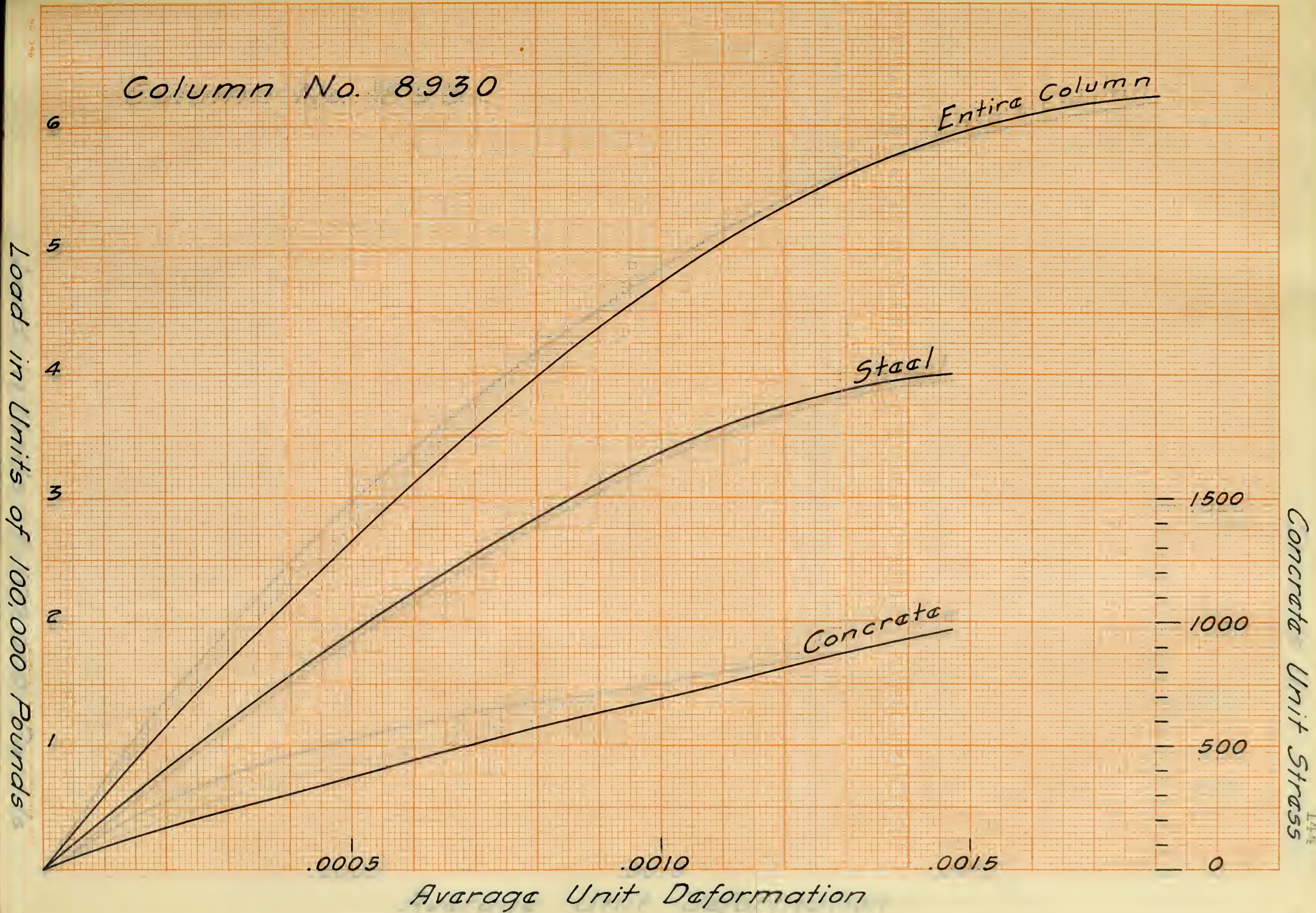


Column No. 8928
Entire Column
Steel
Concrete
6
5
4
3
2
1
.0005
.0010
.0015
2000
1500
1000
500
0
Load in Units of 100,000 Pounds
Average Unit Deformation
Concrete Unit Stress.

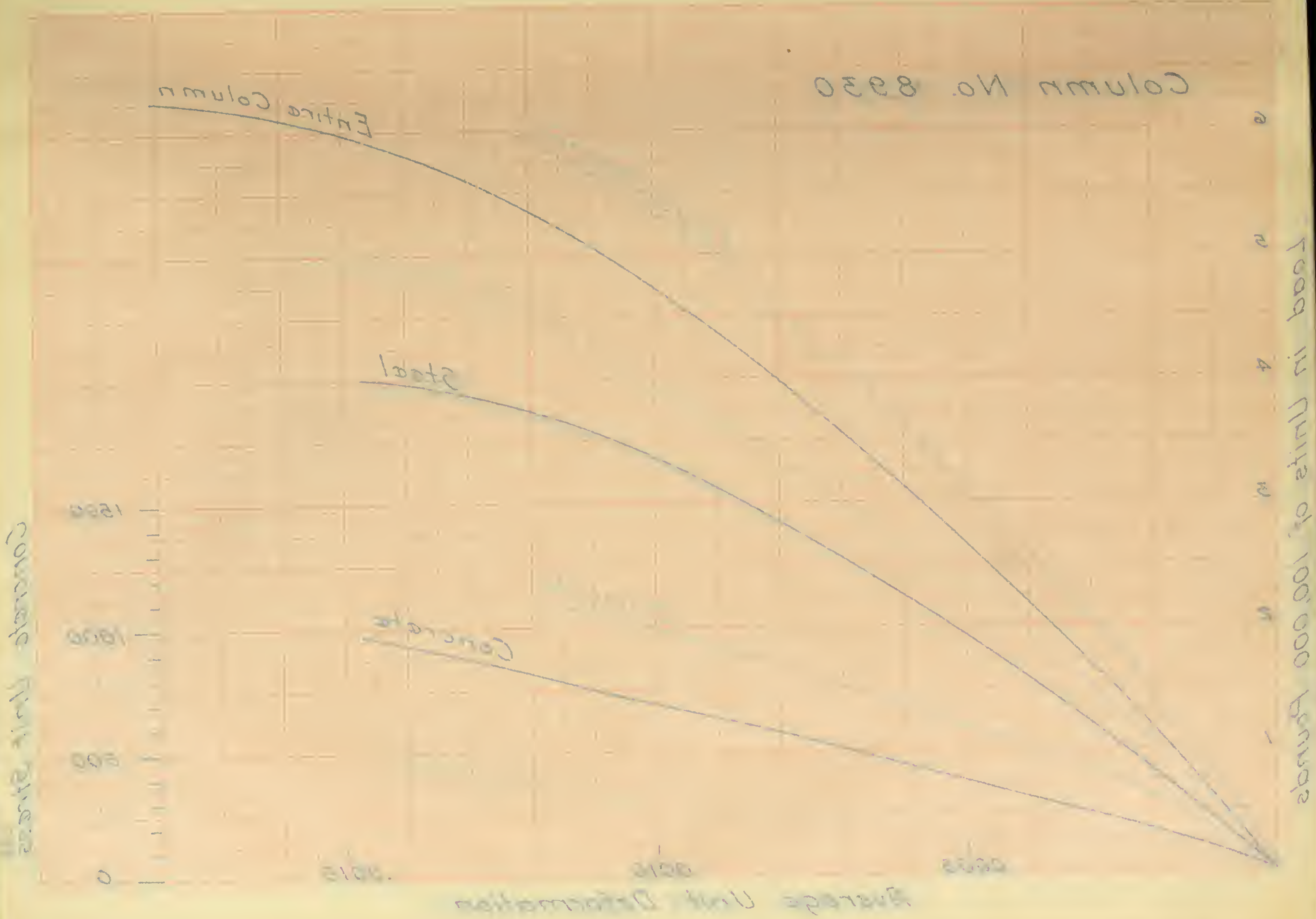

Concrete Unit Stress
Average Unit Deformation
Load in Units of 100,000 Pounds
Concrete
Steel
Entire Column

Column No. 8929
Entire Column.
Steel
Concrete
6
5
4
3
2
1
.0005
.0010
.0015
1500
1000
500
0
Load in Units of 100,000 Pounds
Average Unit Deformation
Concrete Unit Stress
NO 348

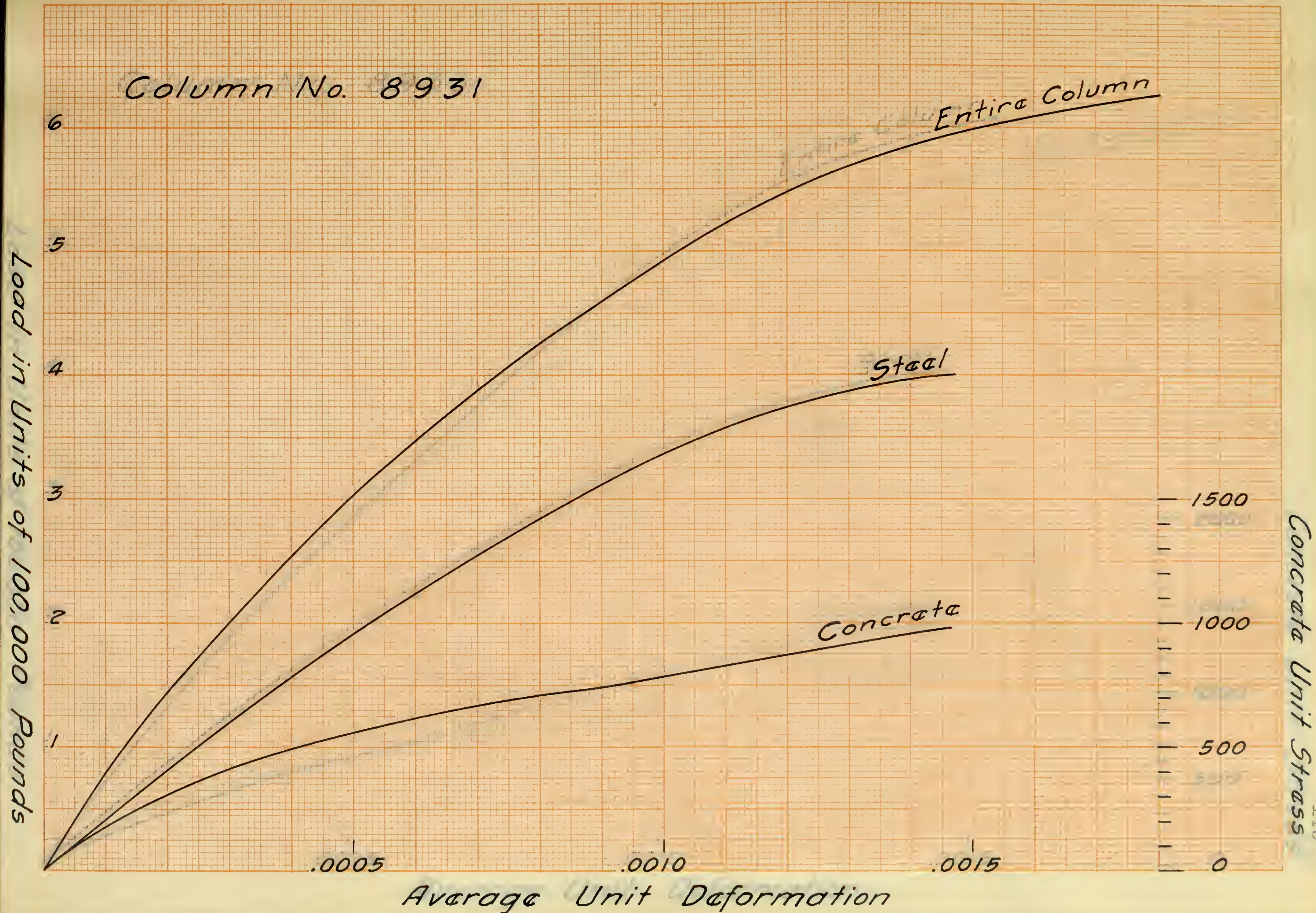

Column No. 8929
Entire Column
Steel
Concrete
Concrete Unit Stress
0
500
1000
1500
Average Unit Deformation
.0005
.0010
.0015
Load in Units of 100,000 Pounds
1
2
3
4
5
6

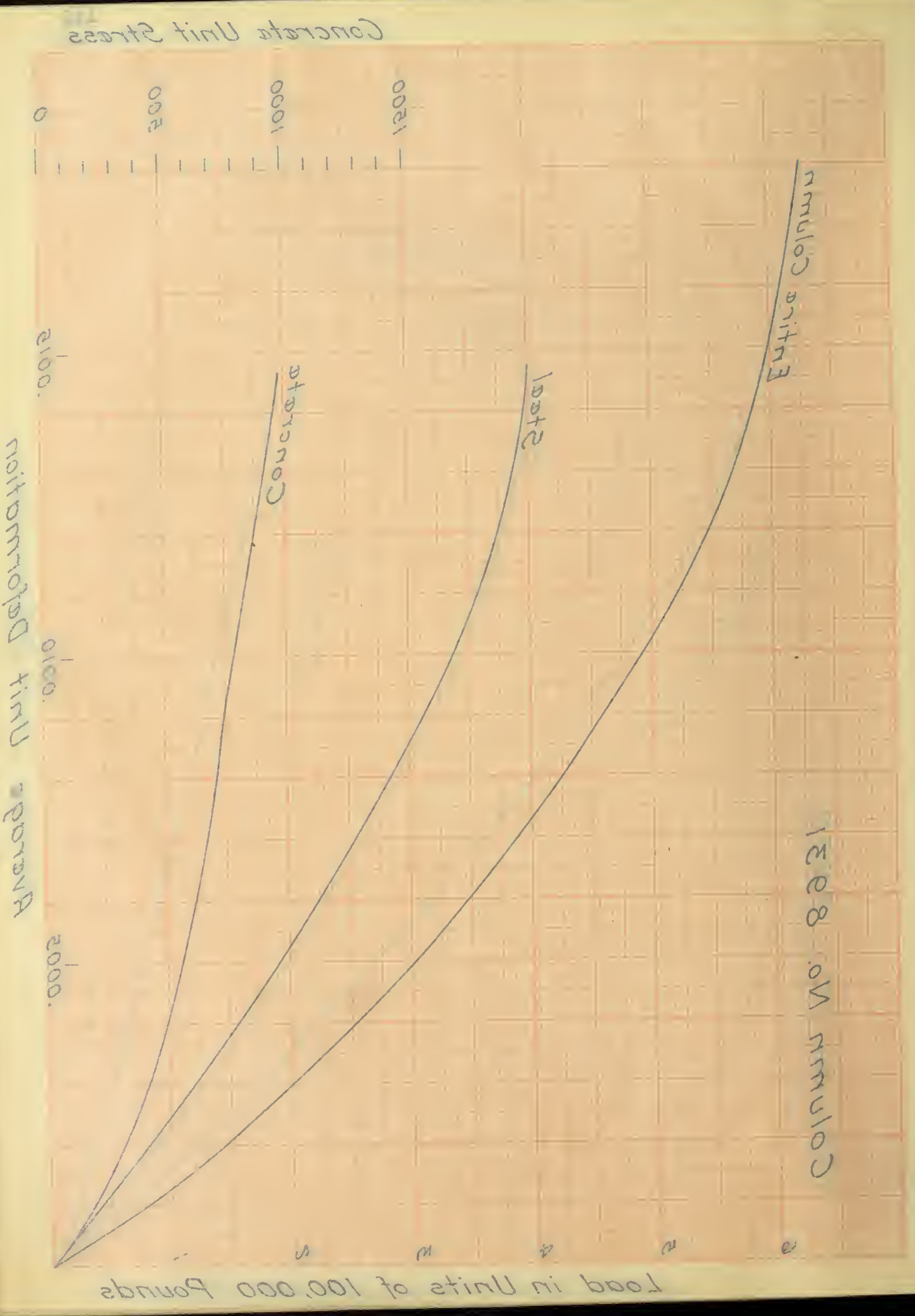


Column No. 8930
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
6
5
4
3
2
1
.0005
.0010
.0015
Average Unit Deformation
Concrete Unit Stress
1500
1000
500
0

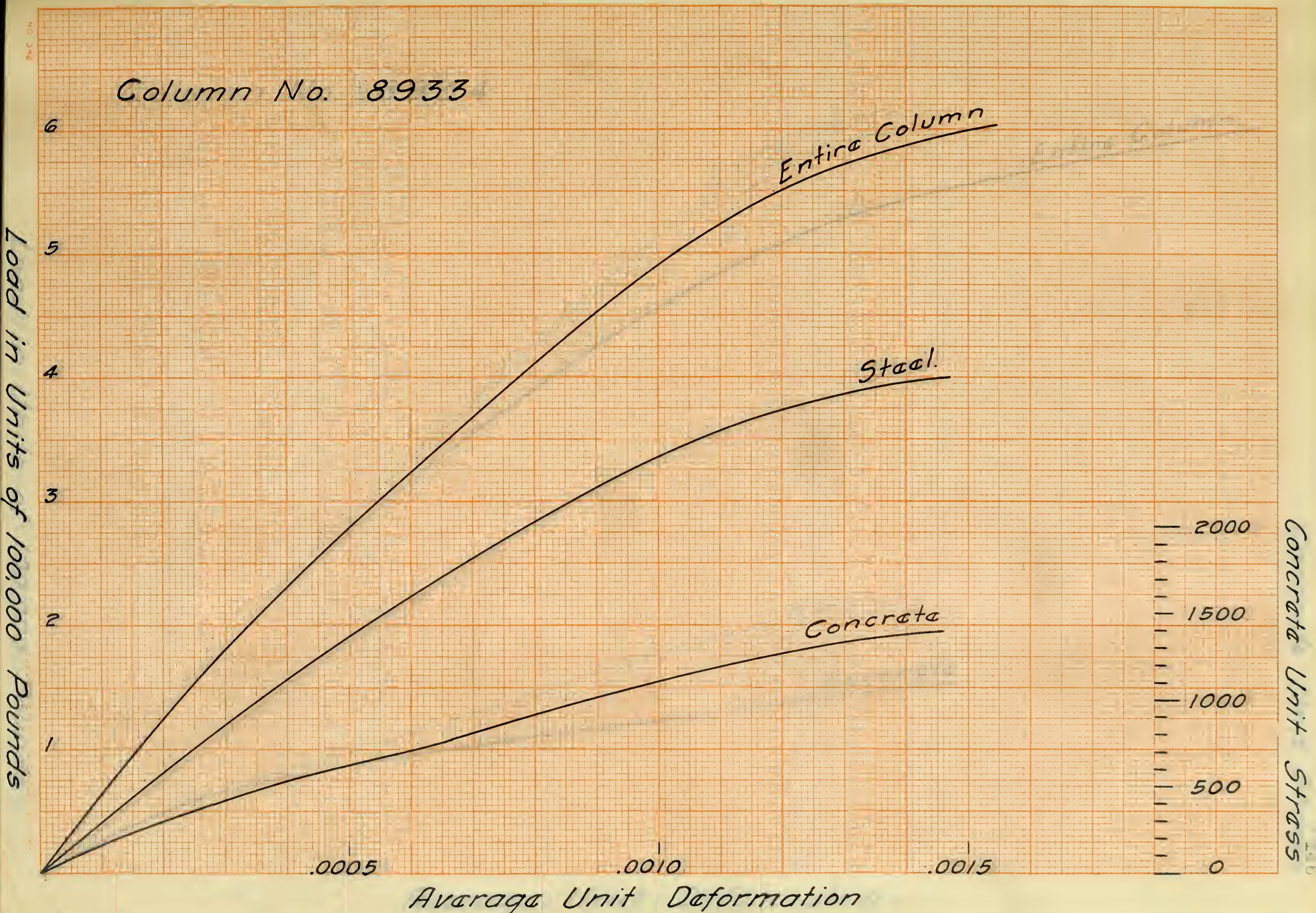


Column No. 8930
Load in Units of 100,000 Pounds
Concrete Unit Stress
Average Unit Deformation
Entire Column
Steel
Concrete

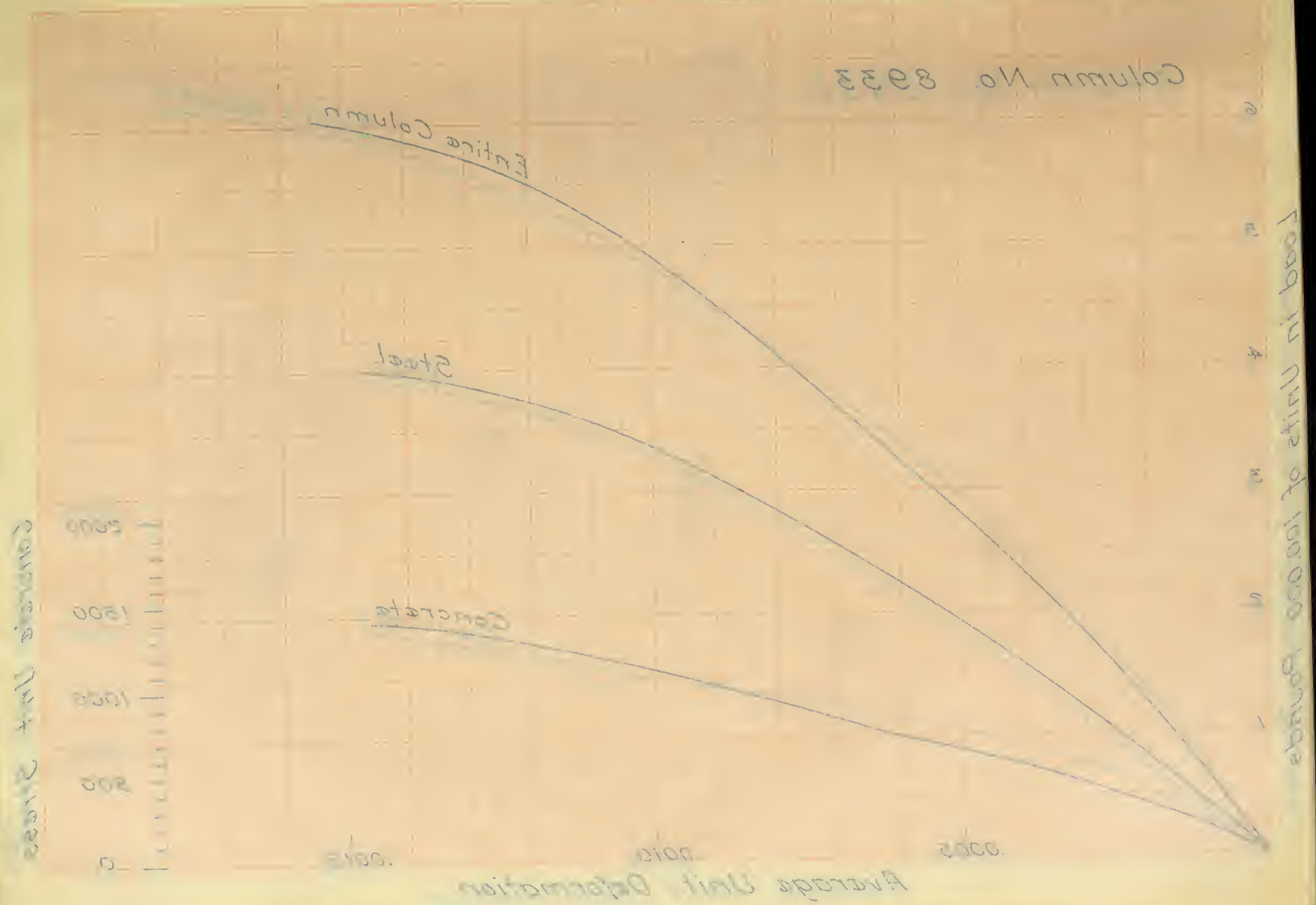

Column No. 8931
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
1
2
3
4
5
6
Average Unit Deformation
.0005
.0010
.0015
Concrete Unit Stress
0
500
1000
1500



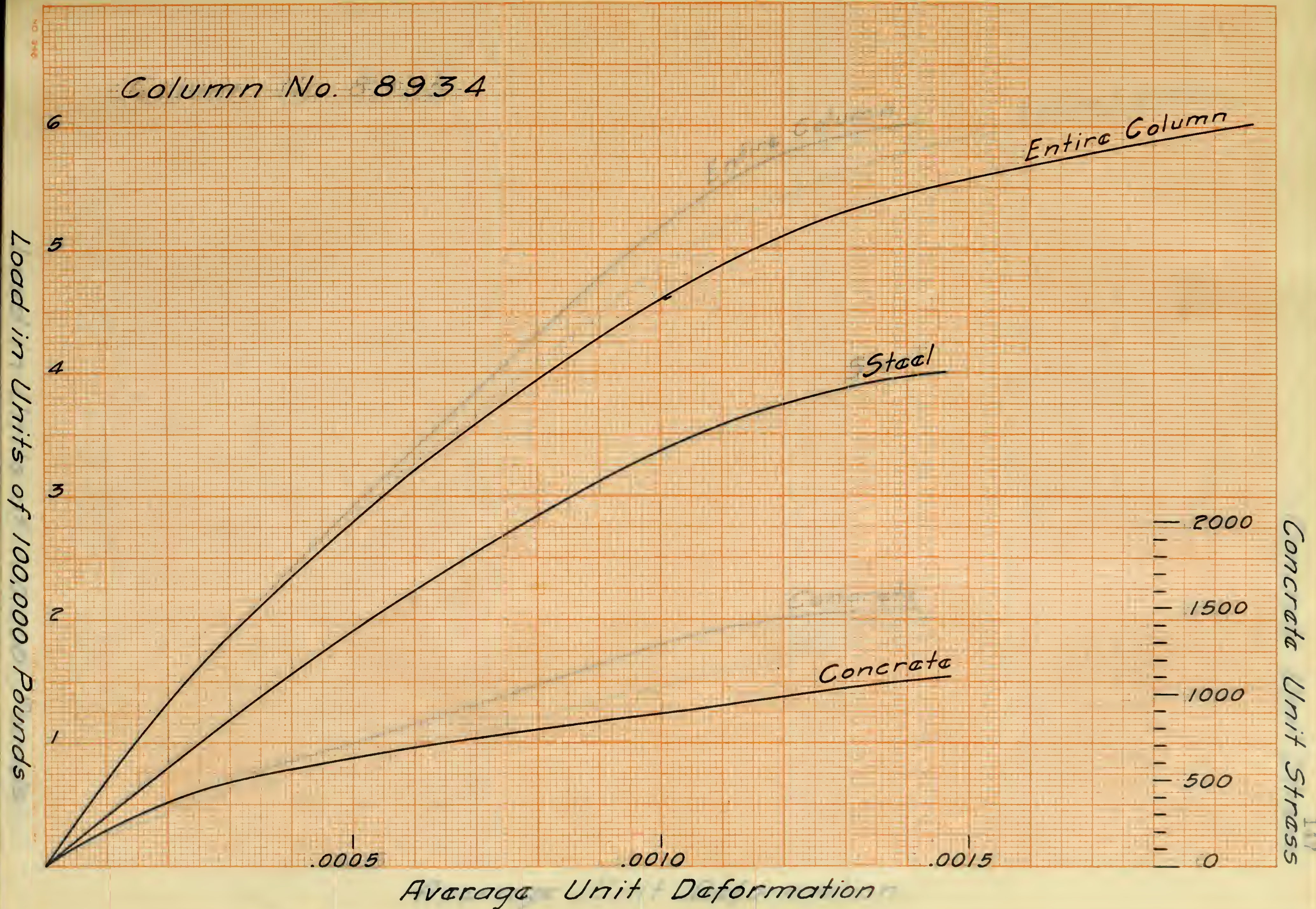
Concrete Unit Stress
0
500
1000
1500
Column No. 8931
Entire Column
Steel
Concrete
Average Unit Deformation
.0005
.0010
.0015
Load in Units of 100.000 Pounds
2
3
4
5
6



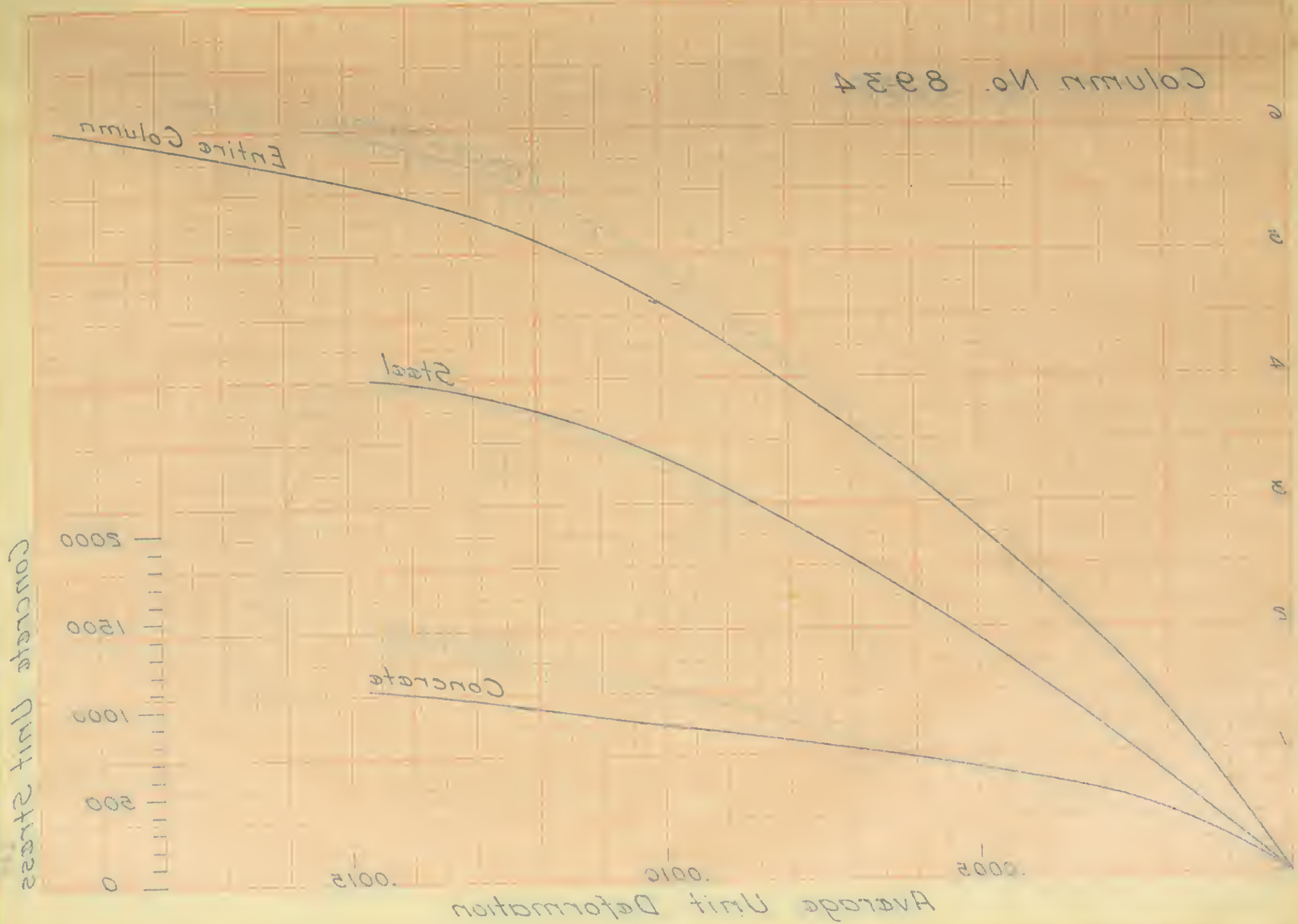
Column No. 8933
Entire Column
Steel.
Concrete
Load in Units of 100,000 Pounds
1
2
3
4
5
6
.0005
.0010
.0015
Average Unit Deformation
Concrete Unit Stress
0
500
1000
1500
2000

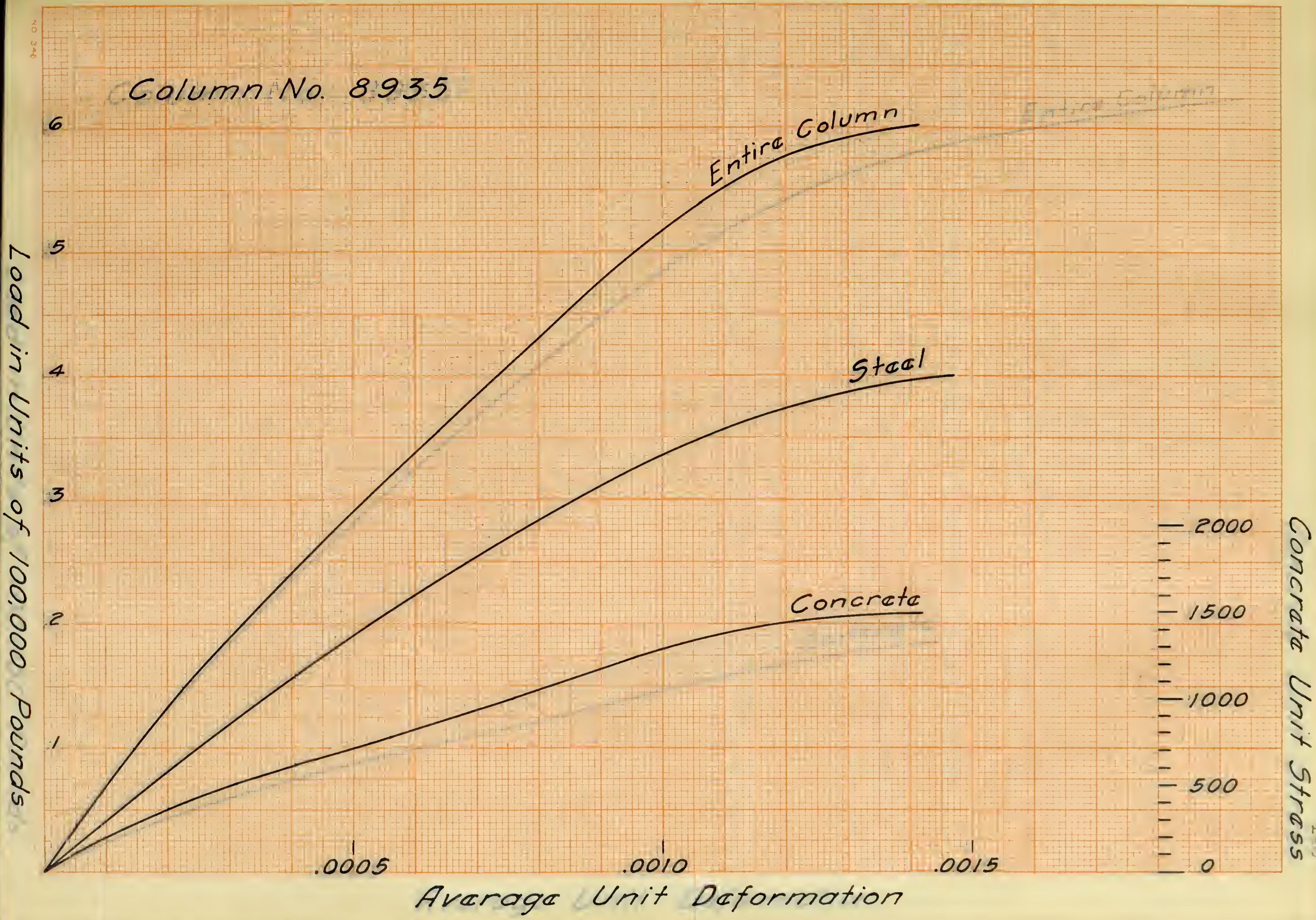


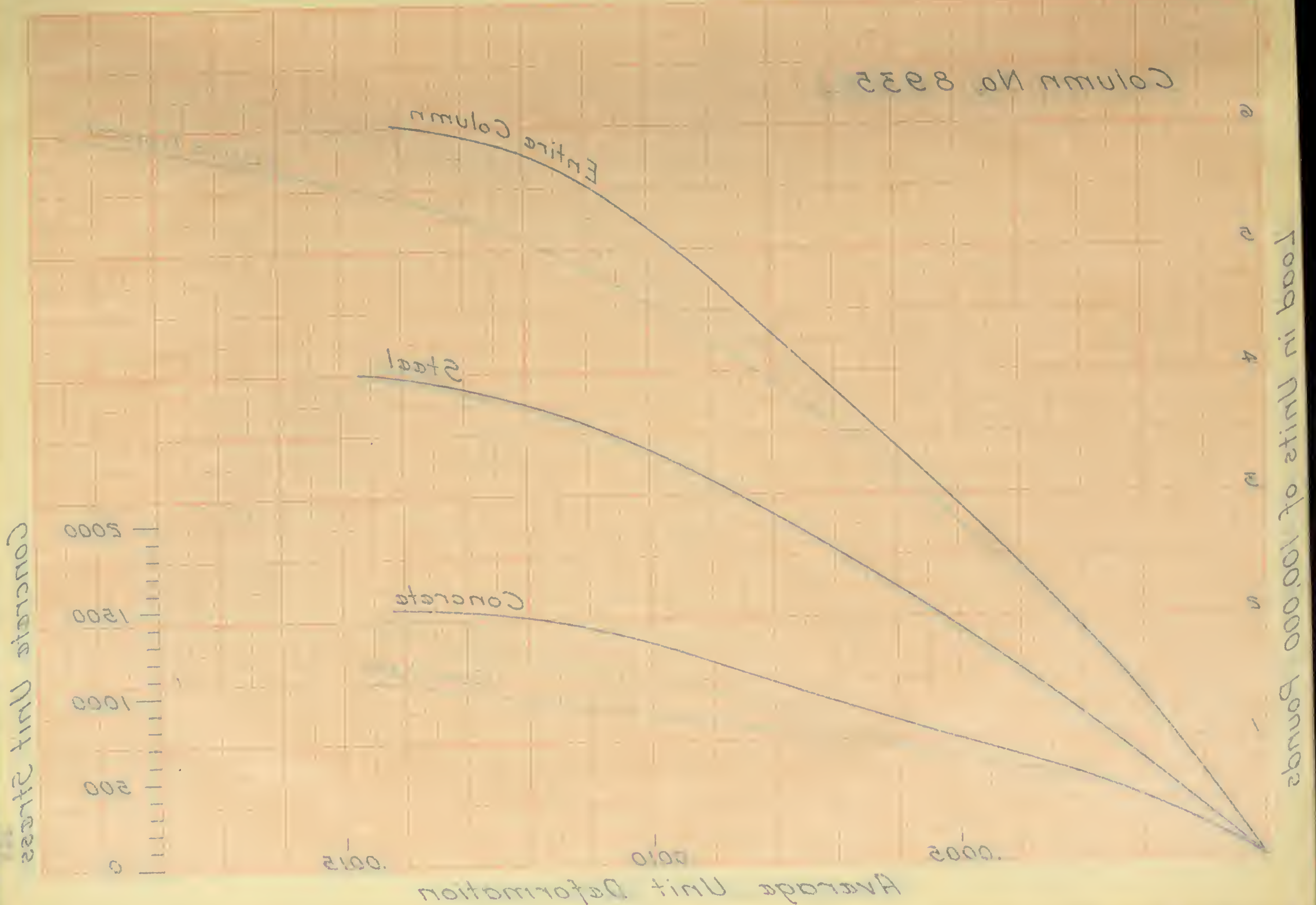
Column No. 8933
Entire Column
Steel
Concrete
Concrete Unit Stress
0
500
1000
1500
2000
Average Unit Deformation
Load in Units of 100,000 Pounds



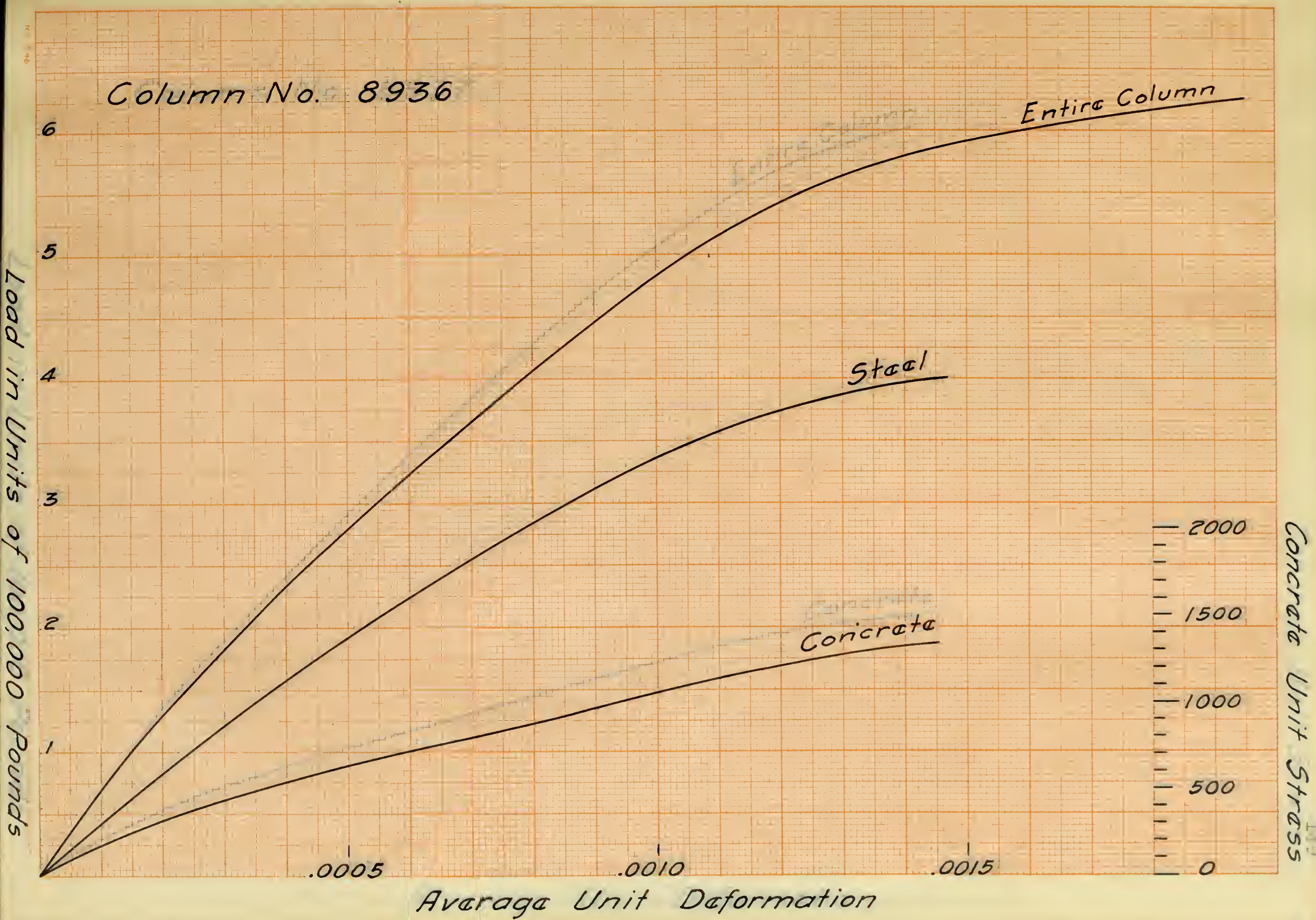
Column No. 8934
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
6
5
4
3
2
1
.0005
.0010
.0015
Average Unit Deformation
Concrete Unit Stress
2000
1500
1000
500
0


Column No. 8934
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
1
2
3
4
5
6
Concrete Unit Stress
0
500
1000
1500
2000
Average Unit Deformation
.0005
.0010
.0015

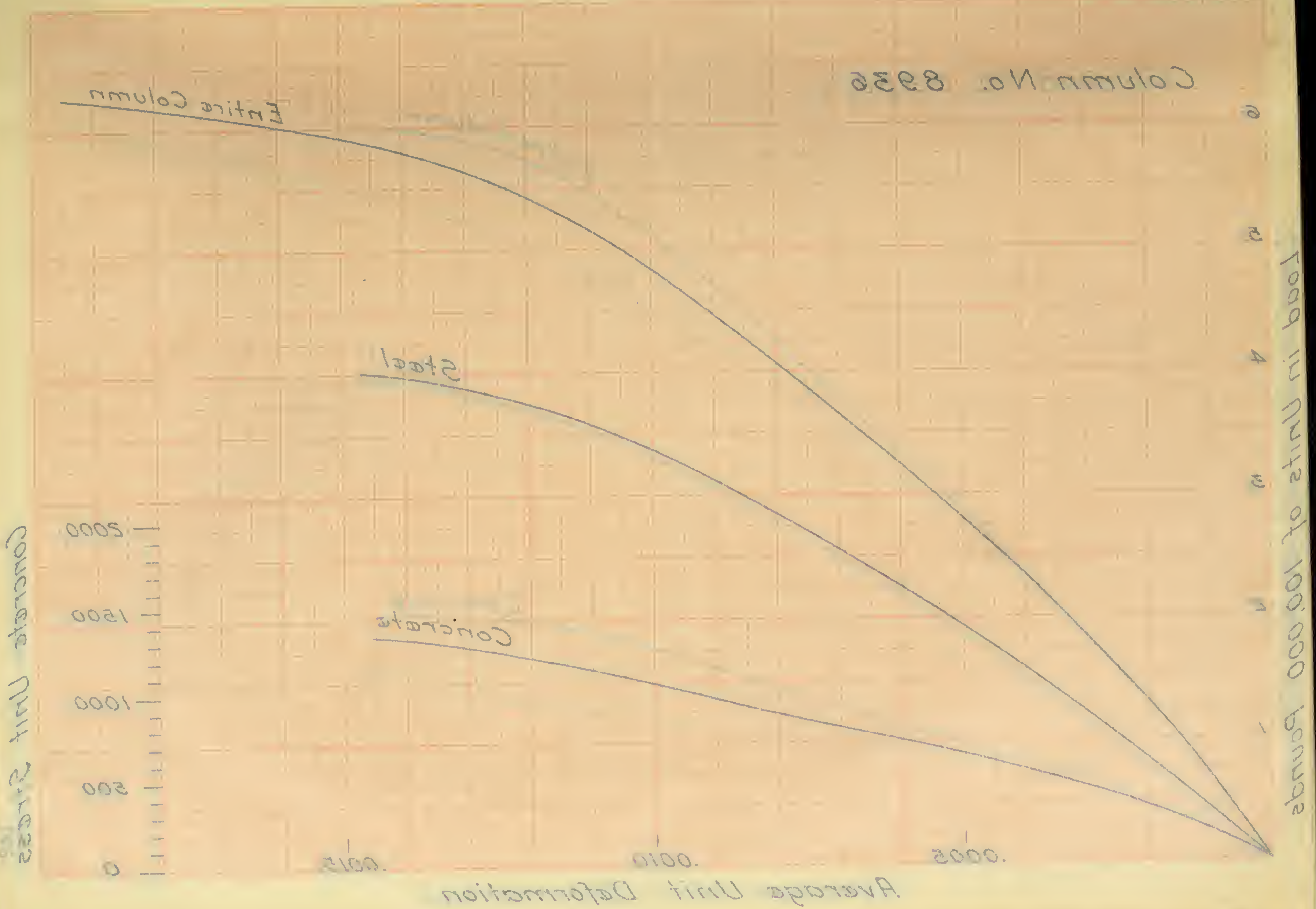

Column No. 8935
Entire Column
Steel
Concrete
6
5
4
3
2
1
.0005
.0010
.0015
2000
1500
1000
500
0
Load in Units of 100,000 Pounds
Average Unit Deformation
Concrete Unit Stress

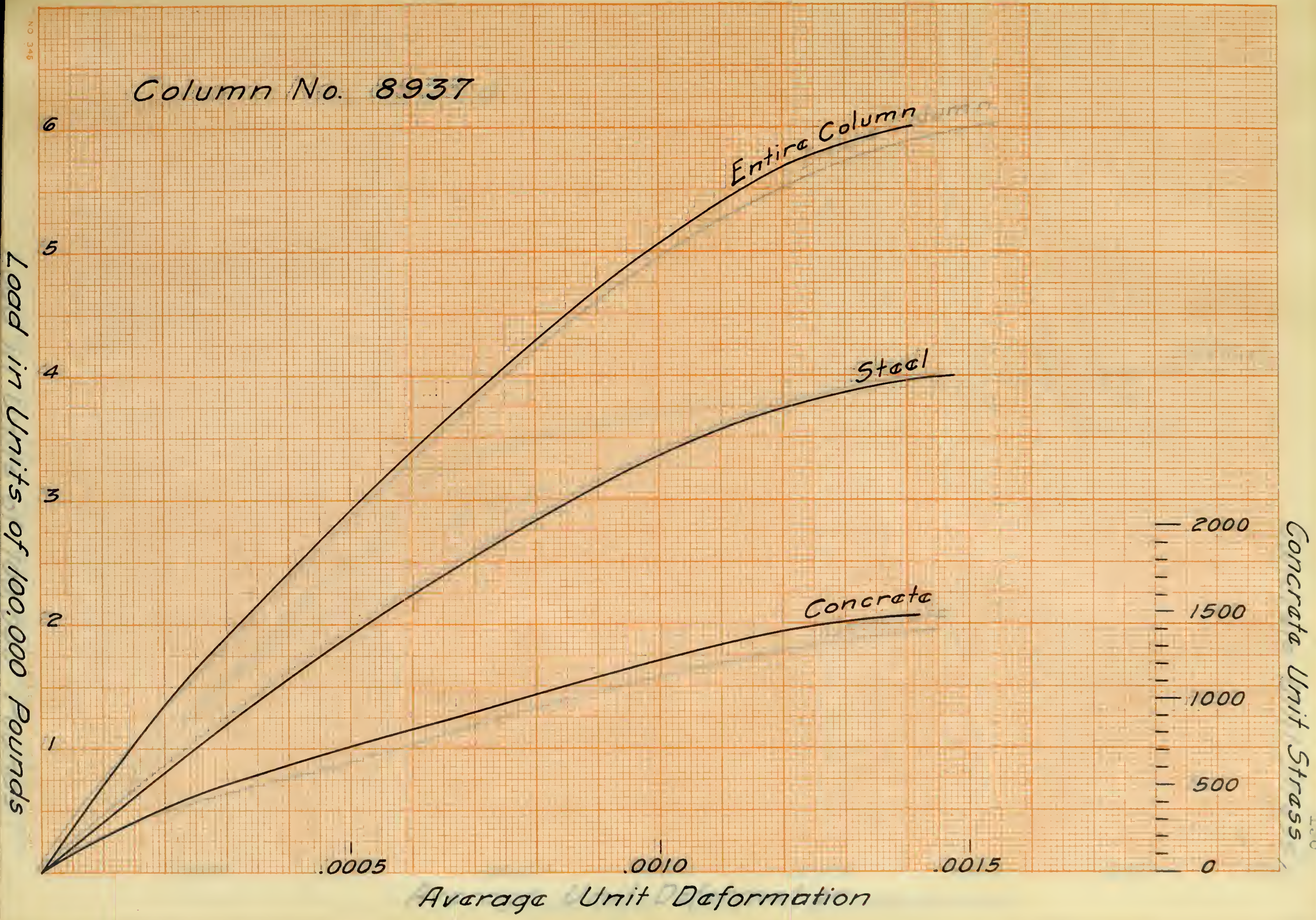


Column No. 8935
Entire Column
Steel
Concrete
Concrete Unit Stress
0
500
1000
1500
2000
Load in Units of 100,000 Pounds
1
2
3
4
5
6
Average Unit Deformation
.0005
.0010
.0015

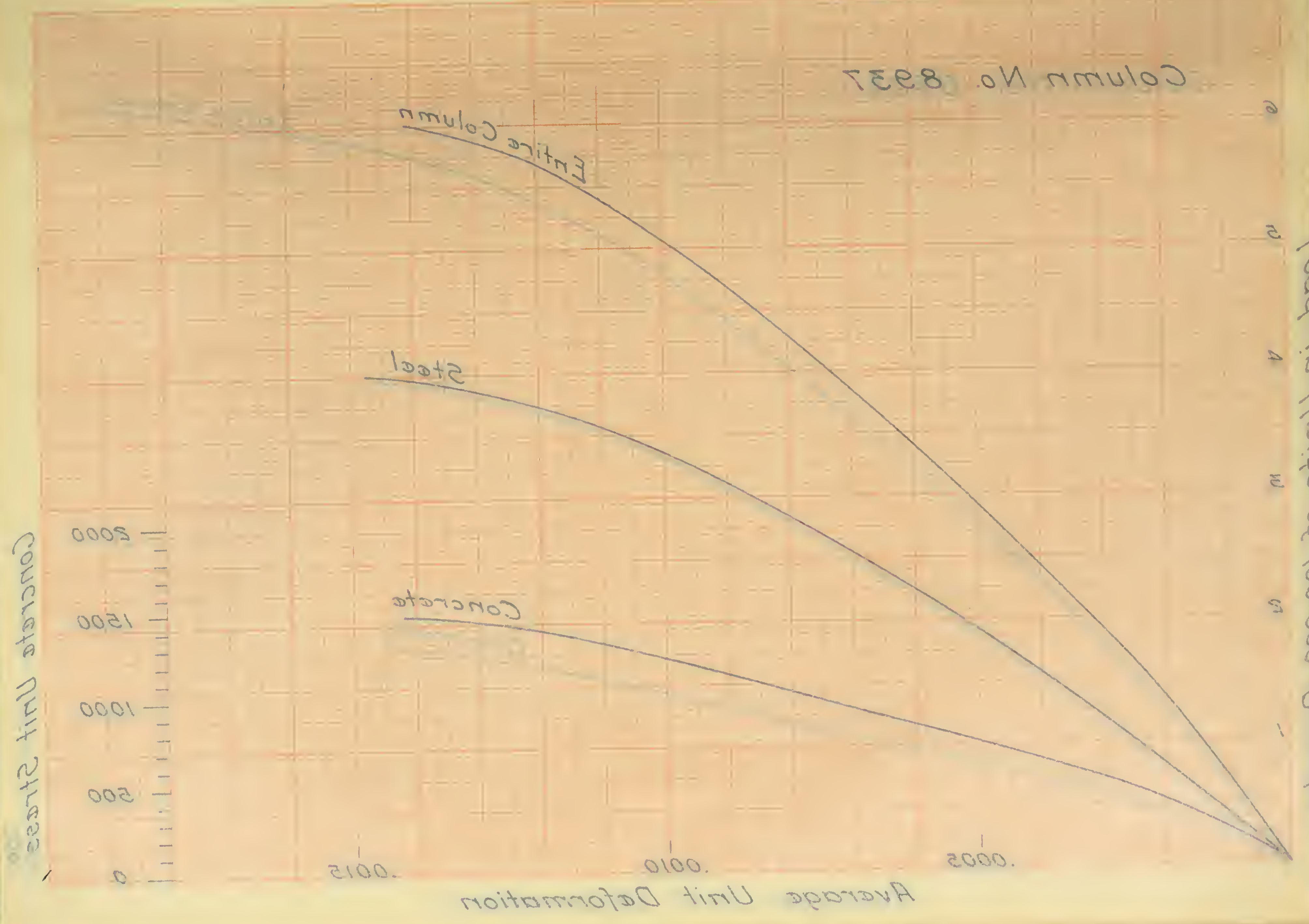


Column No. 8936
Entire Column
Steel
Concrete
6
5
4
3
2
1
.0005
.0010
.0015
2000
1500
1000
500
0
Load in Units of 100,000 Pounds
Average Unit Deformation
Concrete Unit Stress

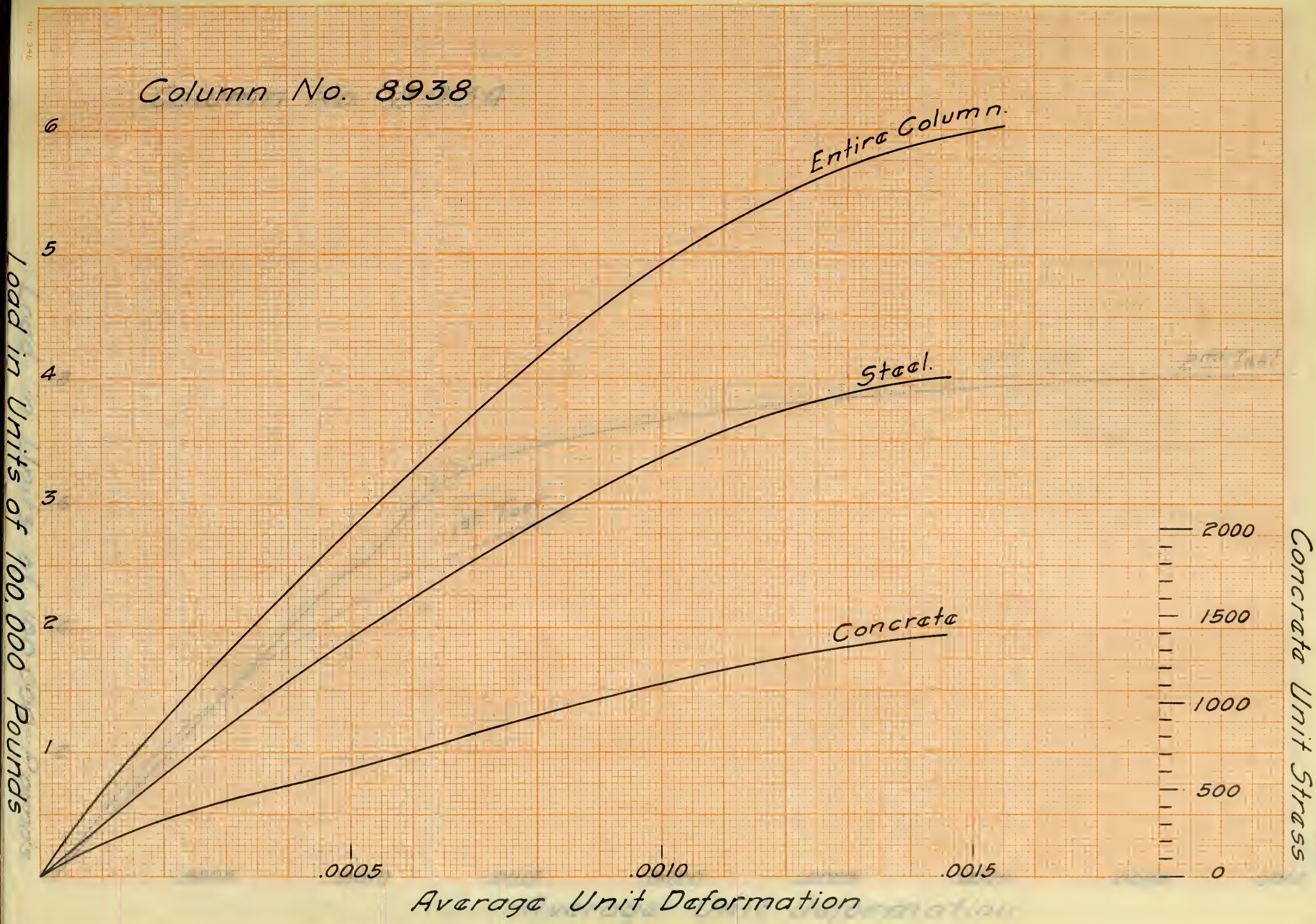

Column No. 8936
Entire Column
Steel
Concrete
Concrete Unit Stress
0
500
1000
1500
2000
Average Unit Deformation
.0005
.0010
.0015
Load in Units of 100,000 Pounds
1
2
3
4
5
6



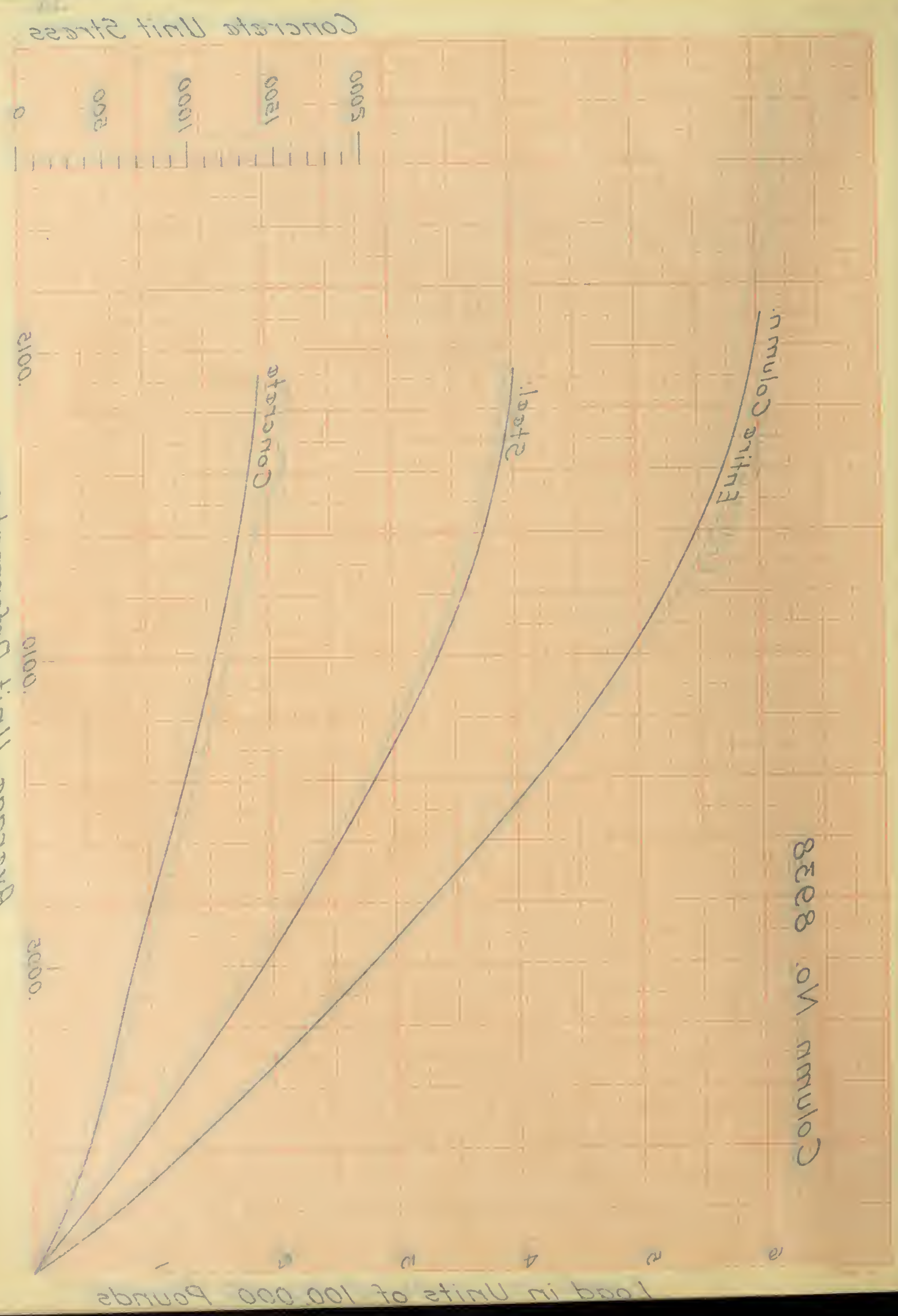
Column No. 8937
Entire Column
Steel
Concrete
Load in Units of 100,000 Pounds
6
5
4
3
2
1
.0005
.0010
.0015
Average Unit Deformation
2000
1500
1000
500
0
Concrete Unit Stress

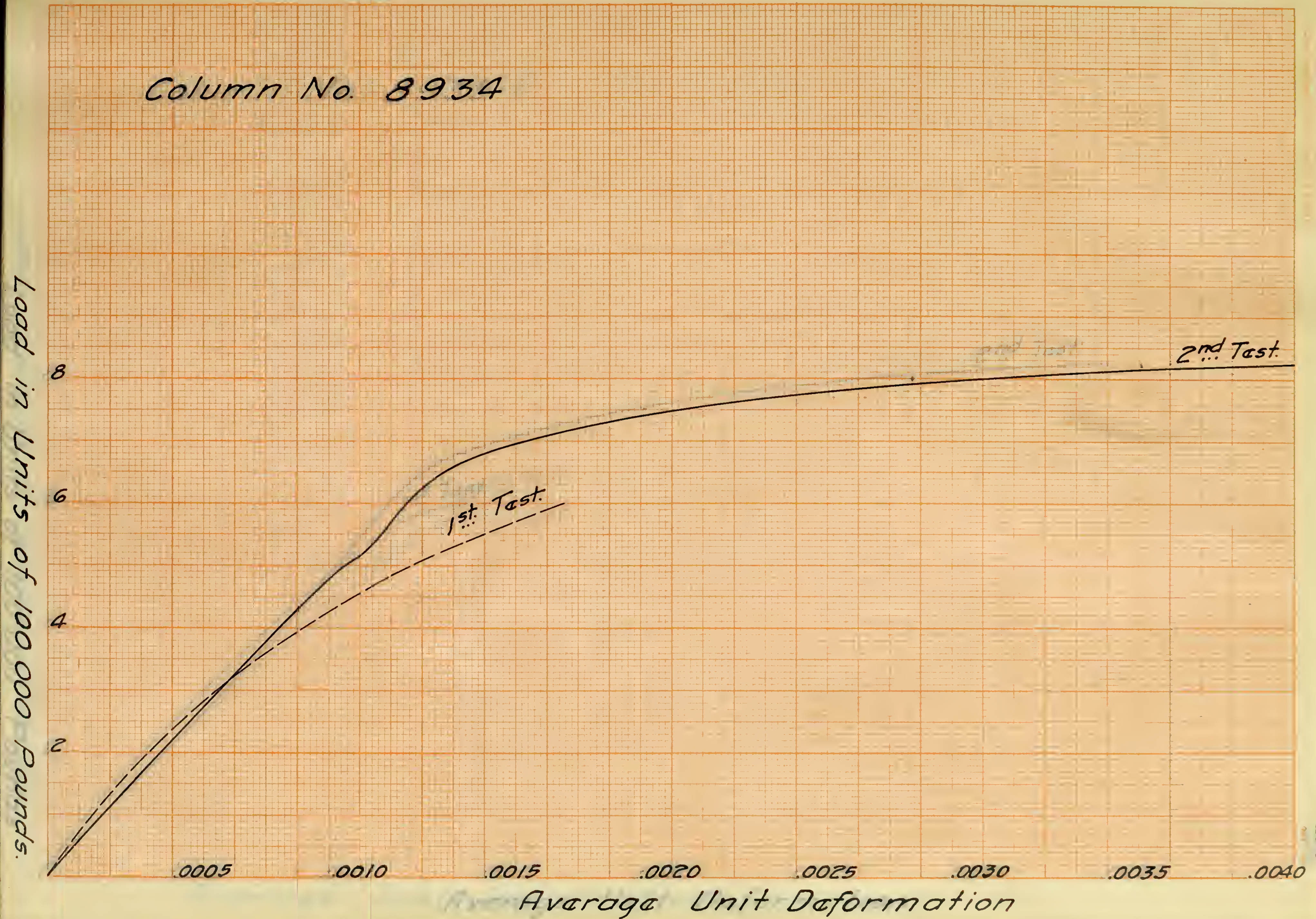


Column No. 8937
Entire Column
Steel
Concrete
Concrete Unit Stress
0
500
1000
1500
2000
Average Unit Deformation
.0005
.0010
.0015
Load in Units of 100.000 Pounds
1
2
3
4
5
6

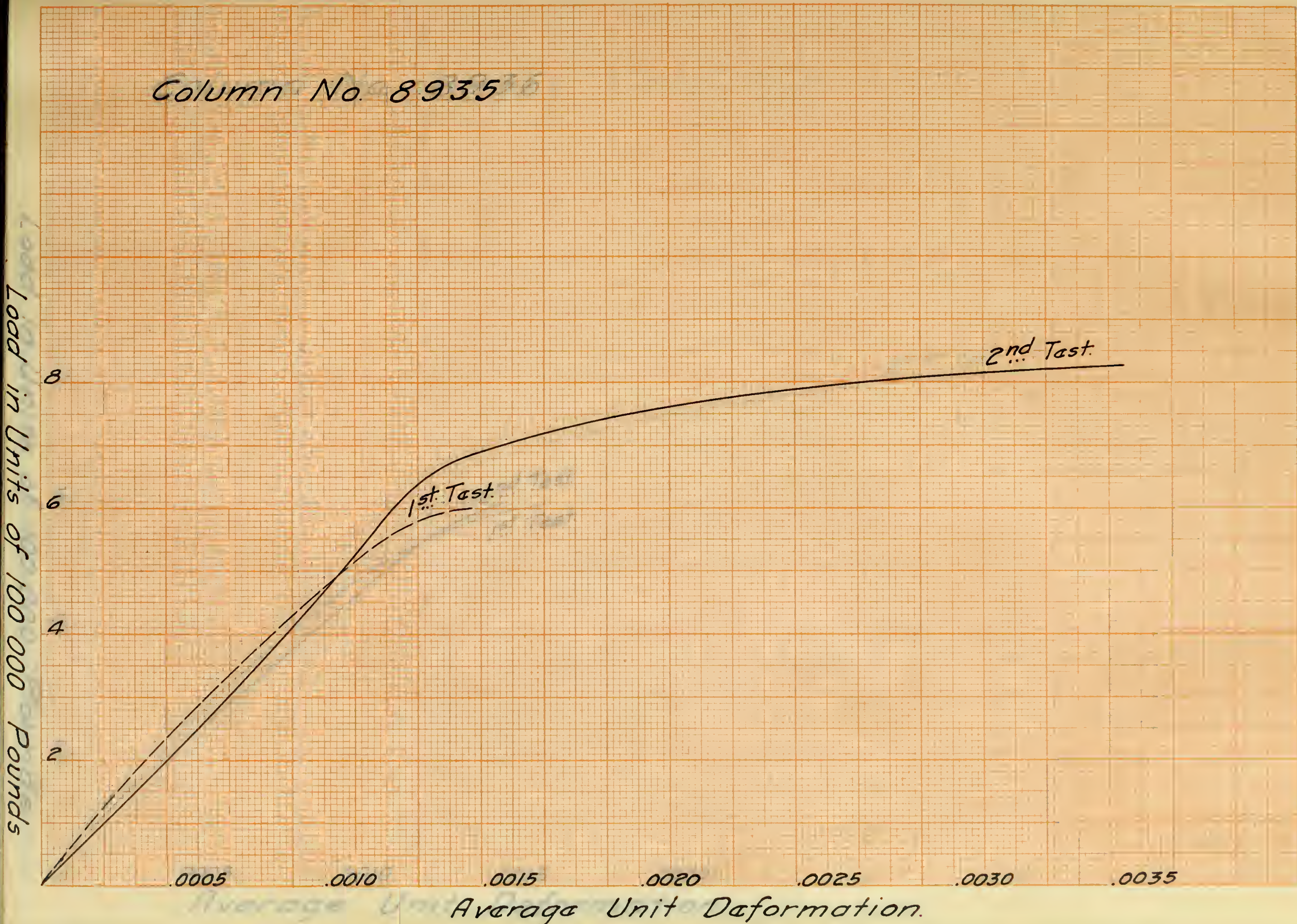


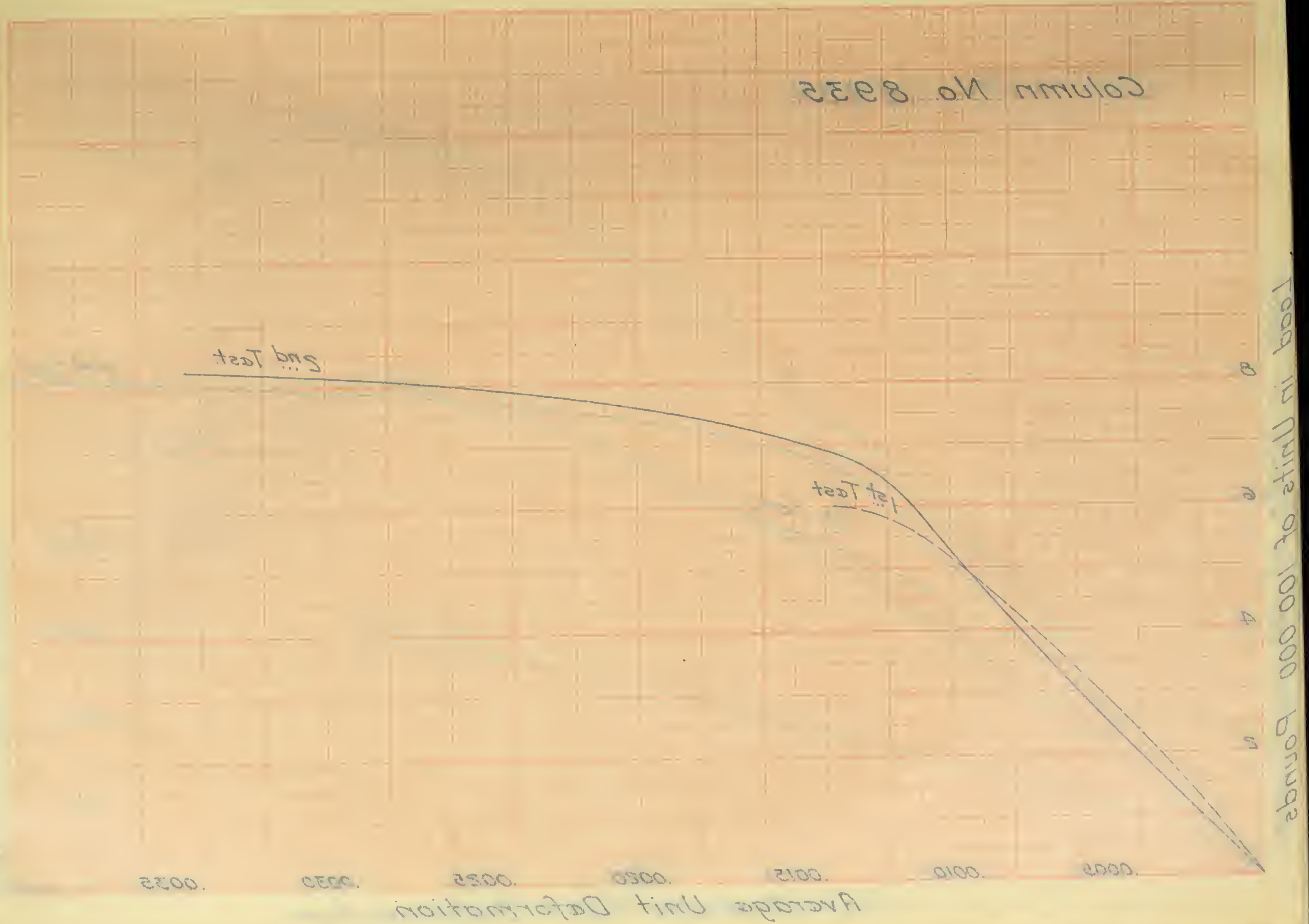
Column No. 8938
Entire Column.
Steel.
Concrete
0
1
2
3
4
5
6
Load in Units of 100,000 Pounds
.0005
.0010
.0015
Average Unit Deformation
0
500
1000
1500
2000
Concrete Unit Stress

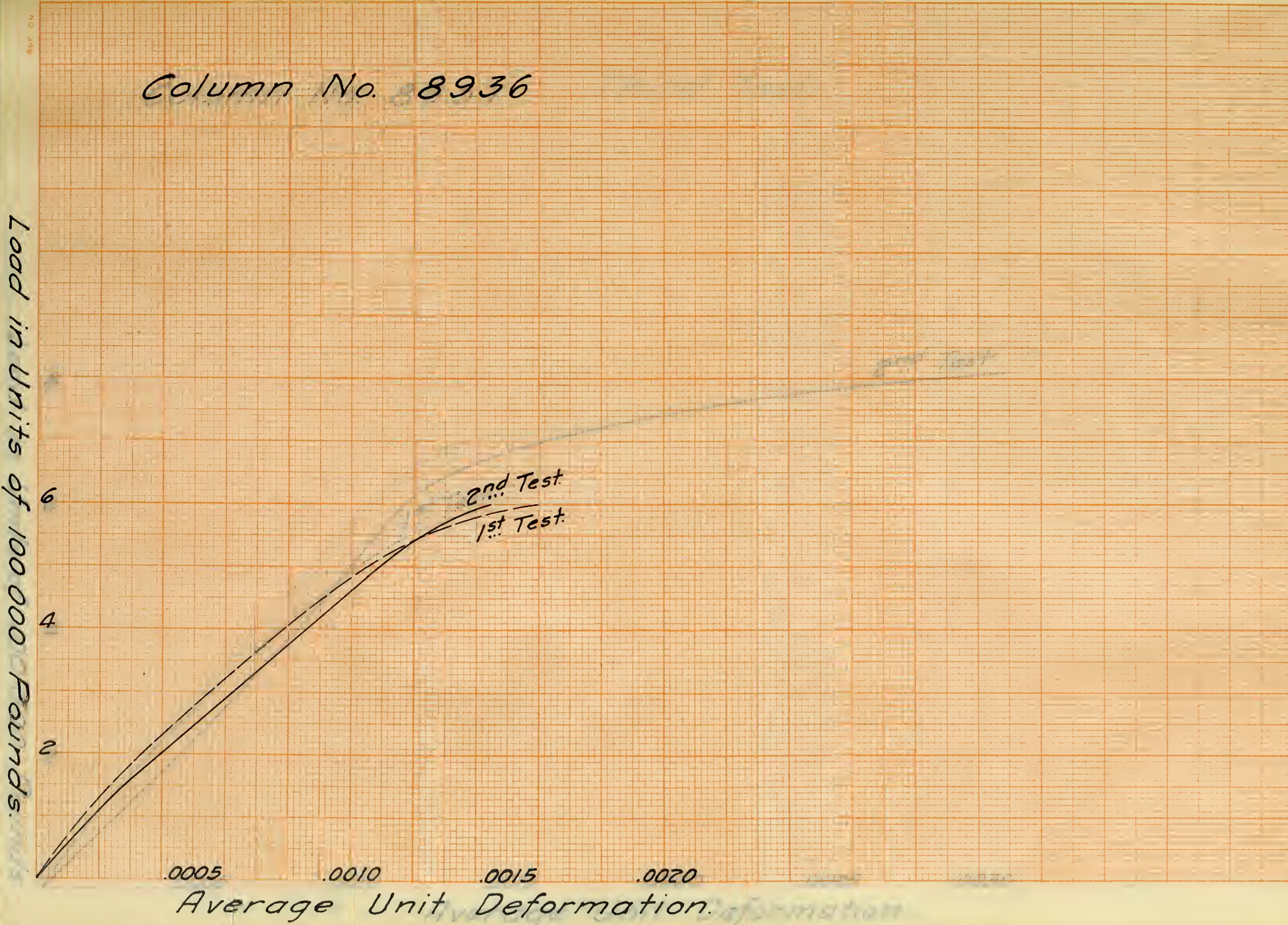

Concrete Unit Stress
0
500
1000
1500
2000
Average Unit Deformation
.0005
.0010
.0015
Concrete
Steel
Entire Column
Column No. 8938
1
2
3
4
5
6
Load in Units of 100,000 Pounds


Column No. 8934
2nd Test.
1st Test.
8
6
4
2
.0005
.0010
.0015
.0020
.0025
.0030
.0035
.0040
Load in Units of 100 000 Pounds.
Average Unit Deformation


Column No. 8934
2nd Test
1st Test
8
6
4
2
.0040
.0035
.0030
.0025
.0020
.0015
.0010
.0005
Average Unit Deformation
Load in Units of 100 000 Pounds.

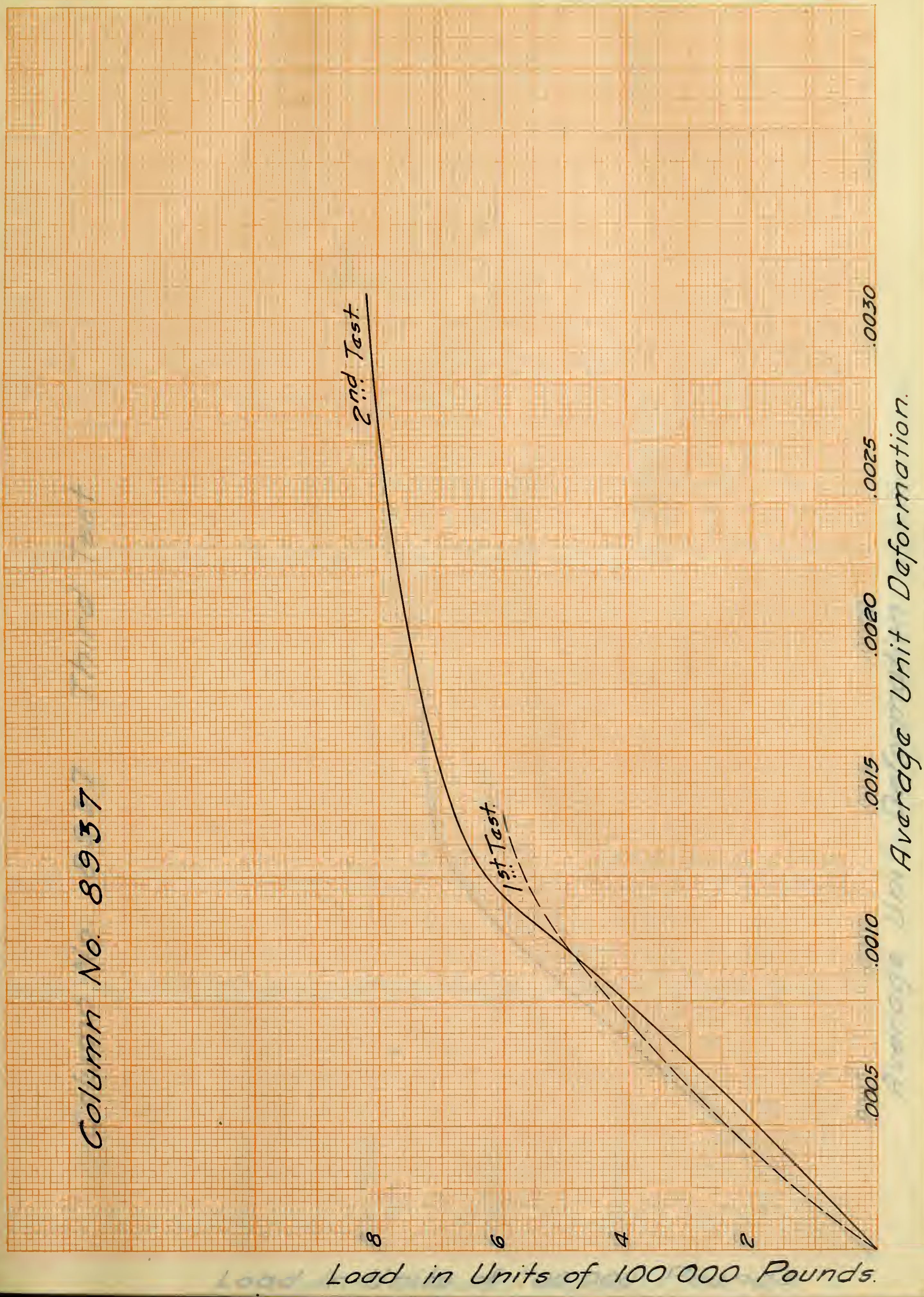

Column No. 8935
2nd Test.
1st. Test.
8
6
4
2
Load in Units of 100 000 Pounds
.0005
.0010
.0015
.0020
.0025
.0030
.0035
Average Unit Deformation.

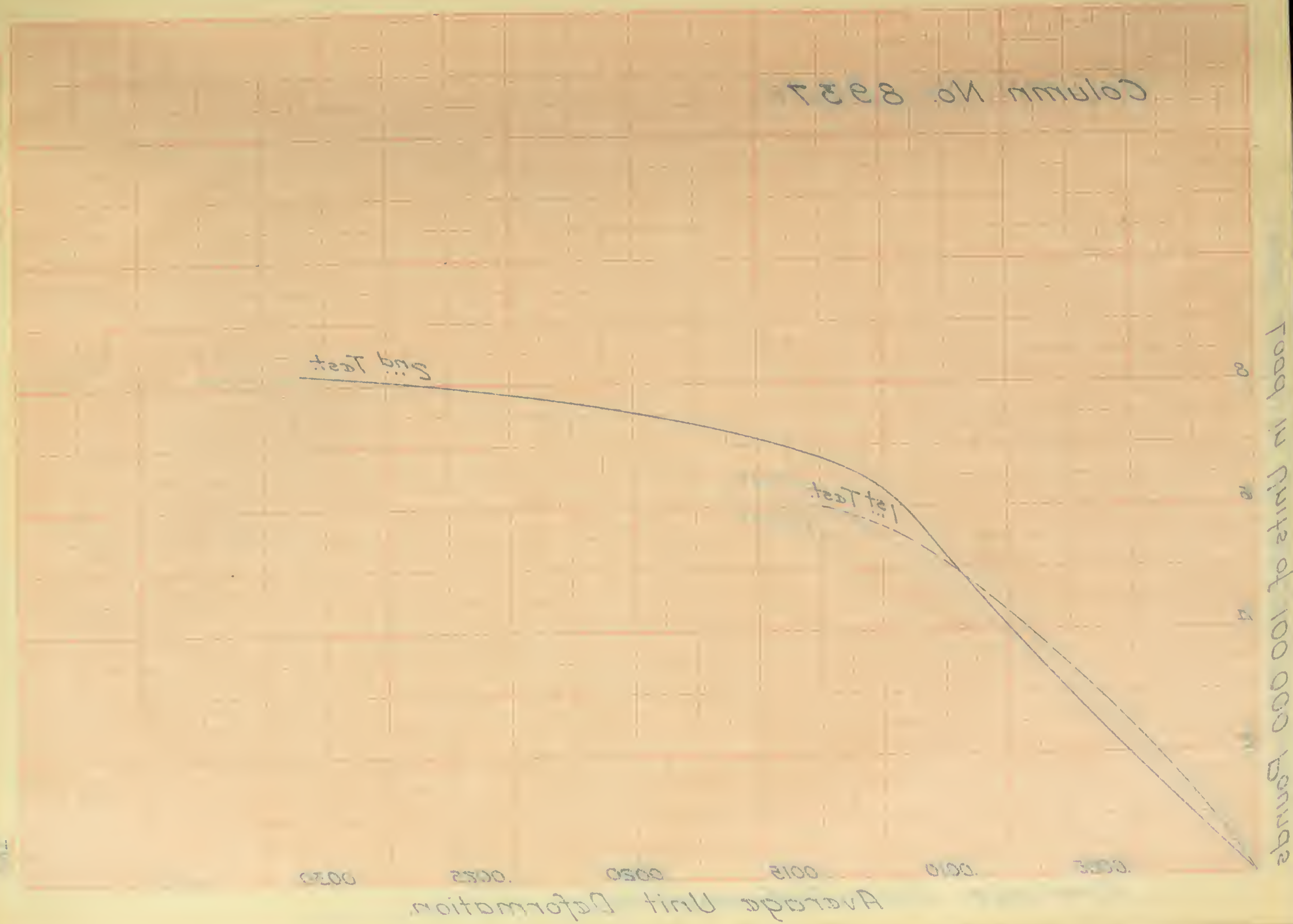

Column No. 8935
2nd Test
1st Test
Load in Units of 100 000 Pounds
8
6
4
2
.0005
.0010
.0015
.0020
.0025
.0030
.0035
Average Unit Deformation

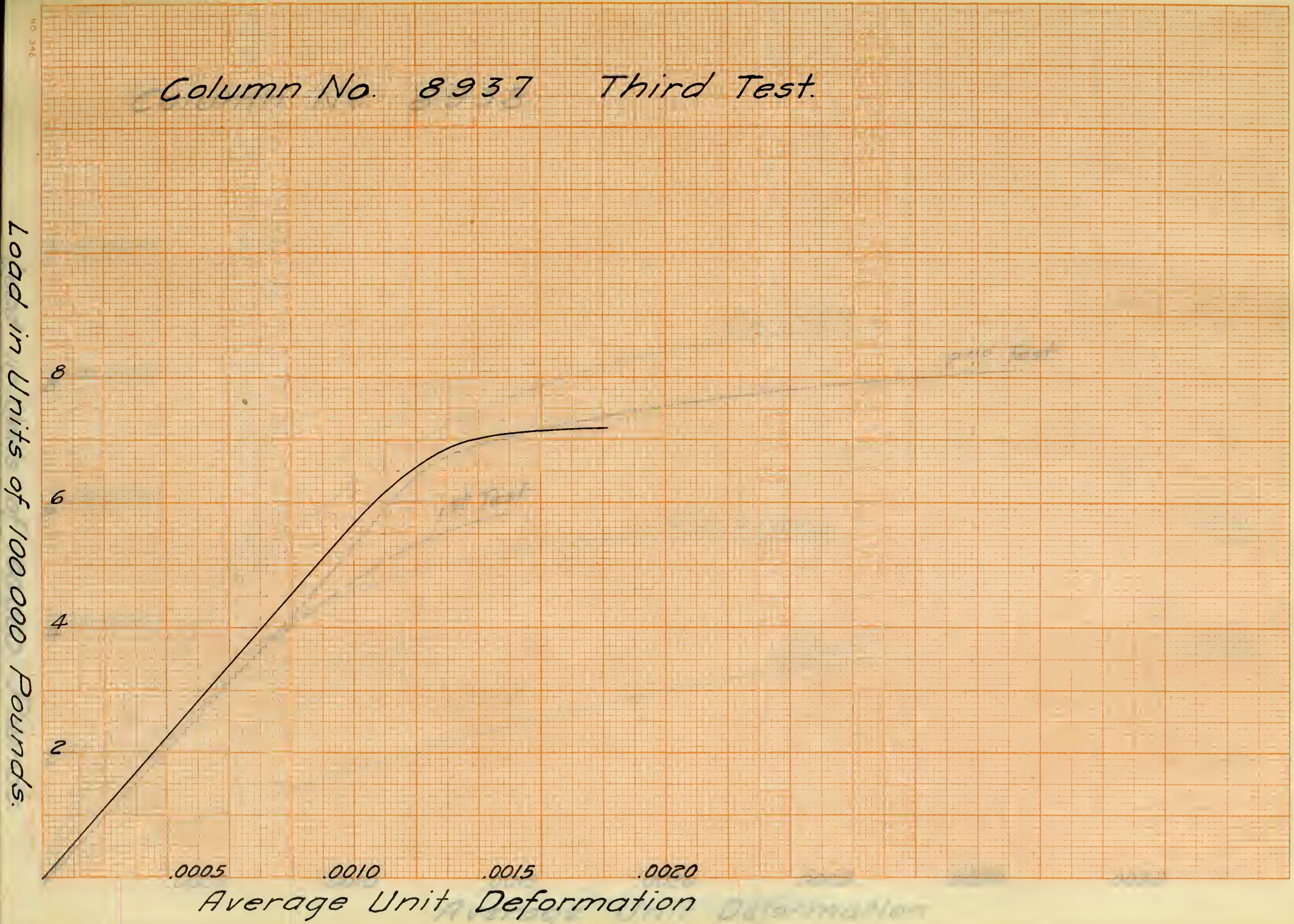


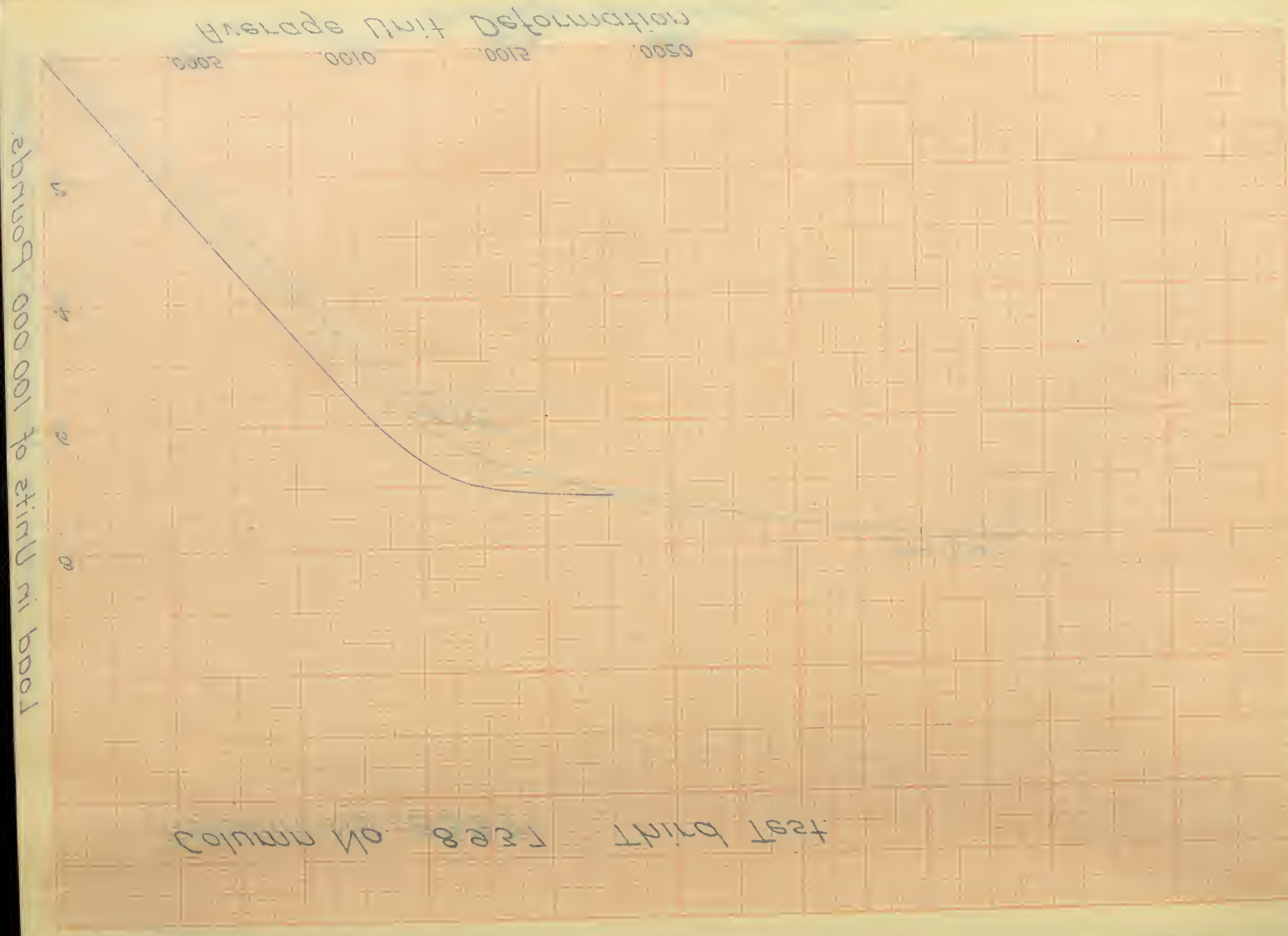
Column No. 8936
2nd Test.
1st Test.
6
4
2
.0005
.0010
.0015
.0020
Average Unit Deformation.
Load in Units of 100 000 Pounds.

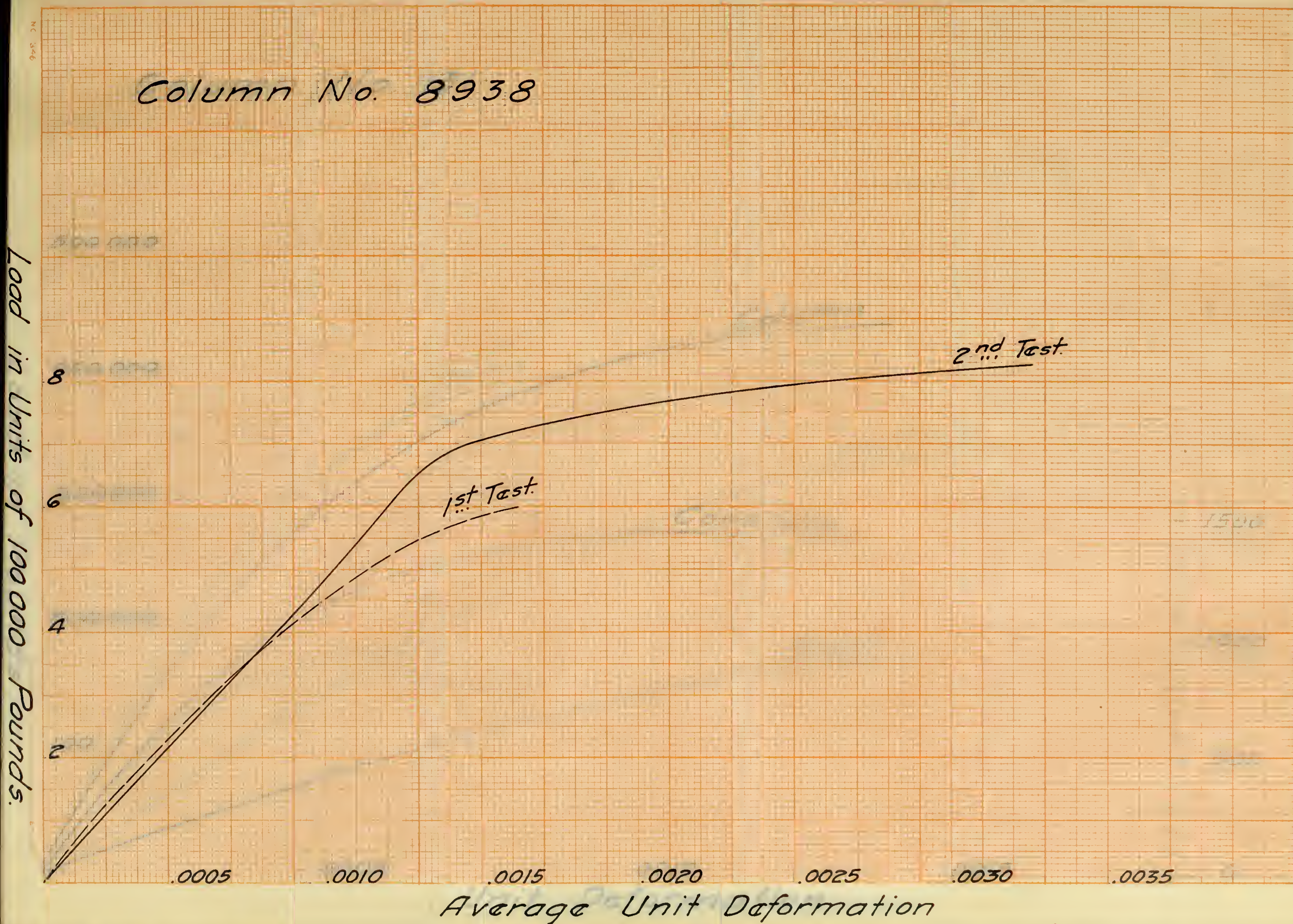
Column No.
1st Test
2nd Test
.0005
.0010
.0015
.0020
Average Unit Deformation
2
4
6
Load in Units of 100 000 Pounds.

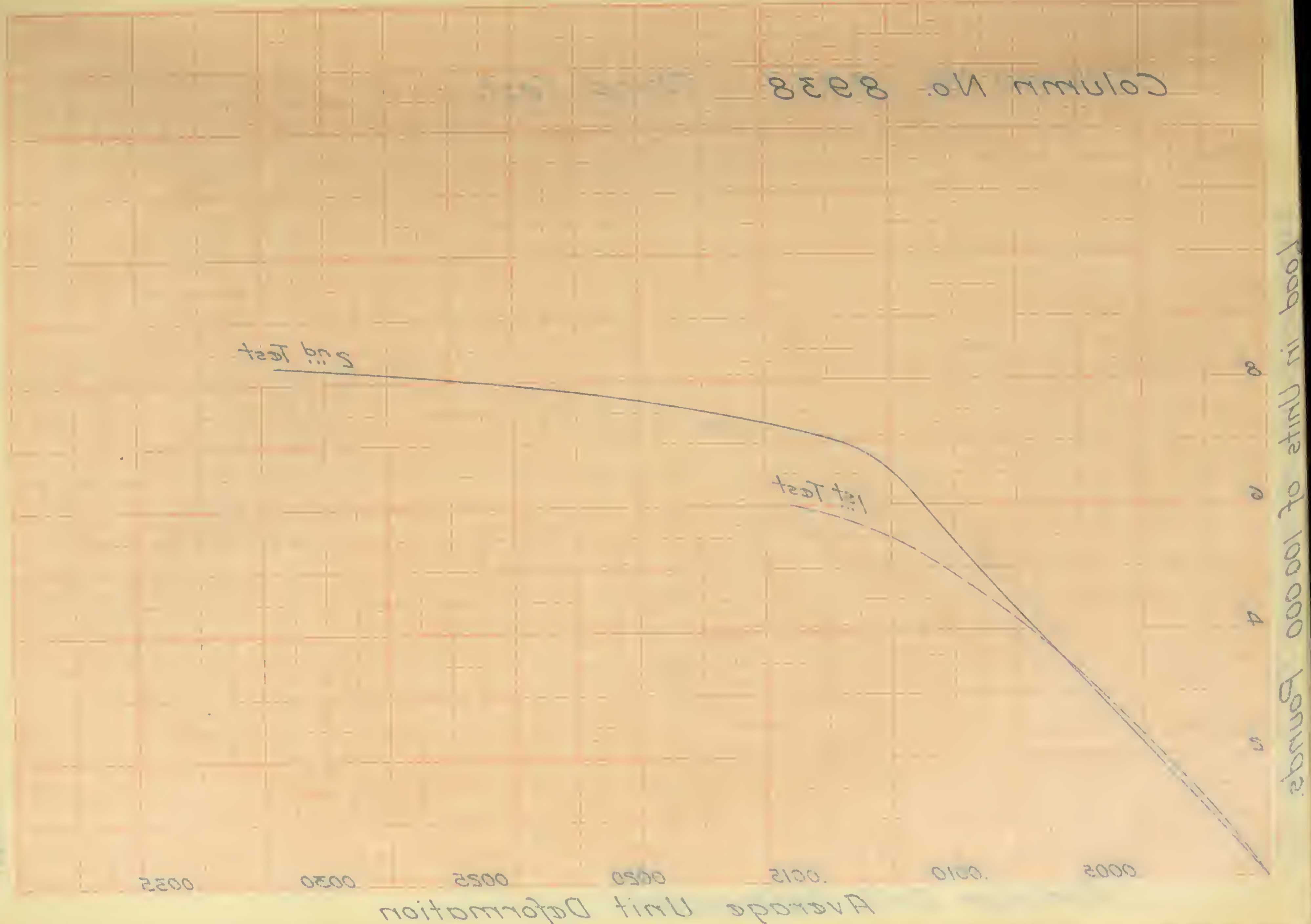

Column No. 8937
2nd Test.
1st Test.
8
6
4
2
.0005
.0010
.0015
.0020
.0025
.0030
Average Unit Deformation.
Load in Units of 100 000 Pounds.


Column No. 8937
2nd Test
1st Test
8
6
4
2
.0005
.0010
.0015
.0020
.0025
.0030
Average Unit Deformation
Load in Units of 100 000 Pounds

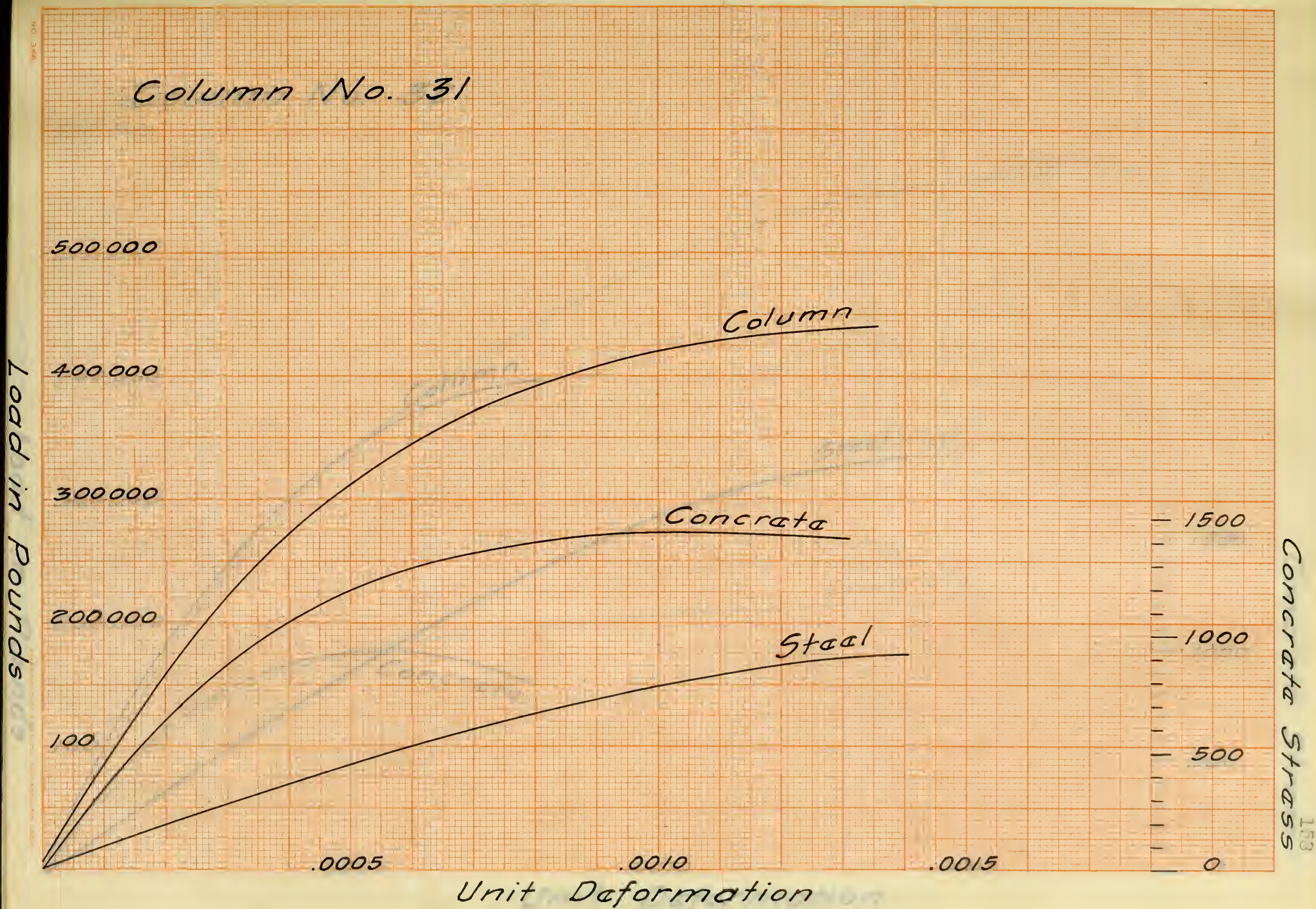

Column No. 8937 Third Test.
8
6
4
2
.0005
.0010
.0015
.0020
Average Unit Deformation
Load in Units of 100 000 Pounds.


Column No. 8937 Third Test
Average Unit Deformation
.0005
.0010
.0015
.0020
Load in Units of 100 000 Pounds.
2
4
6
8

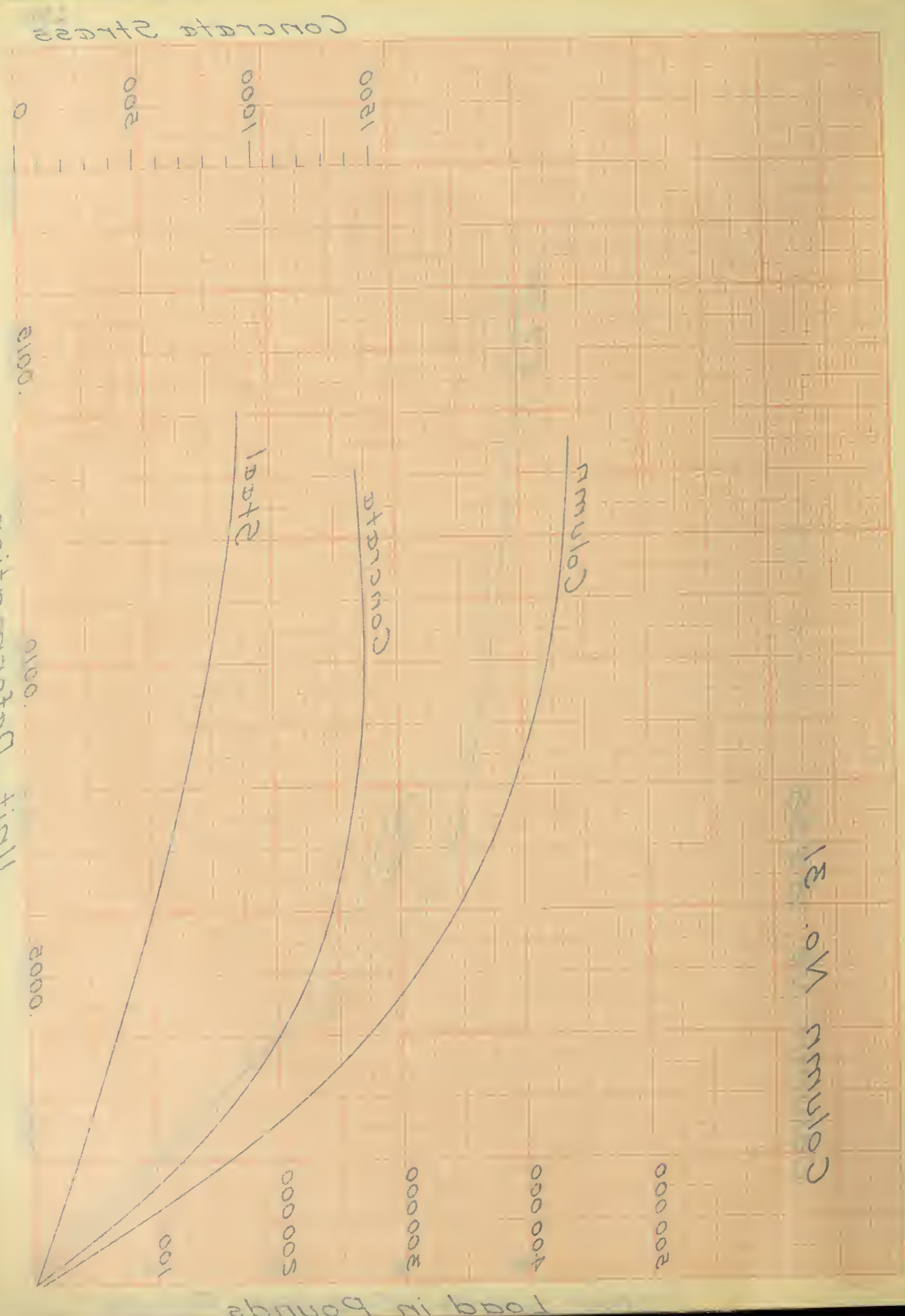

Column No. 8938
2nd Test.
1st Test.
8
6
4
2
.0005
.0010
.0015
.0020
.0025
.0030
.0035
Average Unit Deformation
Load in Units of 100 000 Pounds.



Column No. 8938
2nd Test
1st Test
Load in Units of 100 000 Pounds
2
4
6
8
.0005
.0010
.0015
.0020
.0025
.0030
.0035
Average Unit Deformation

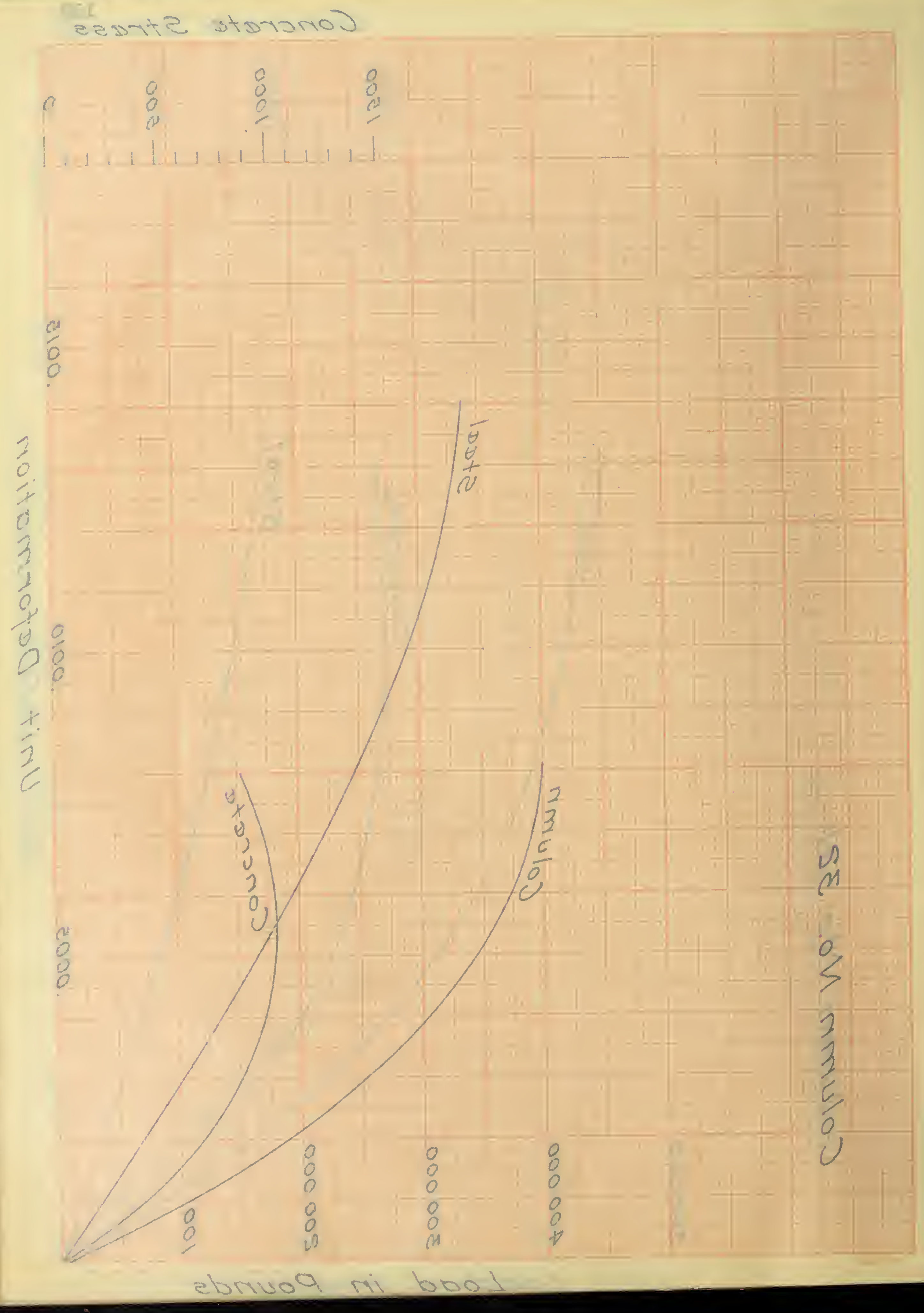


Column No. 31
500000
400000
300000
200000
100
Load in Pounds
Column
Concrete
Steel
.0005
.0010
.0015
Unit Deformation
1500
1000
500
0
Concrete Stress



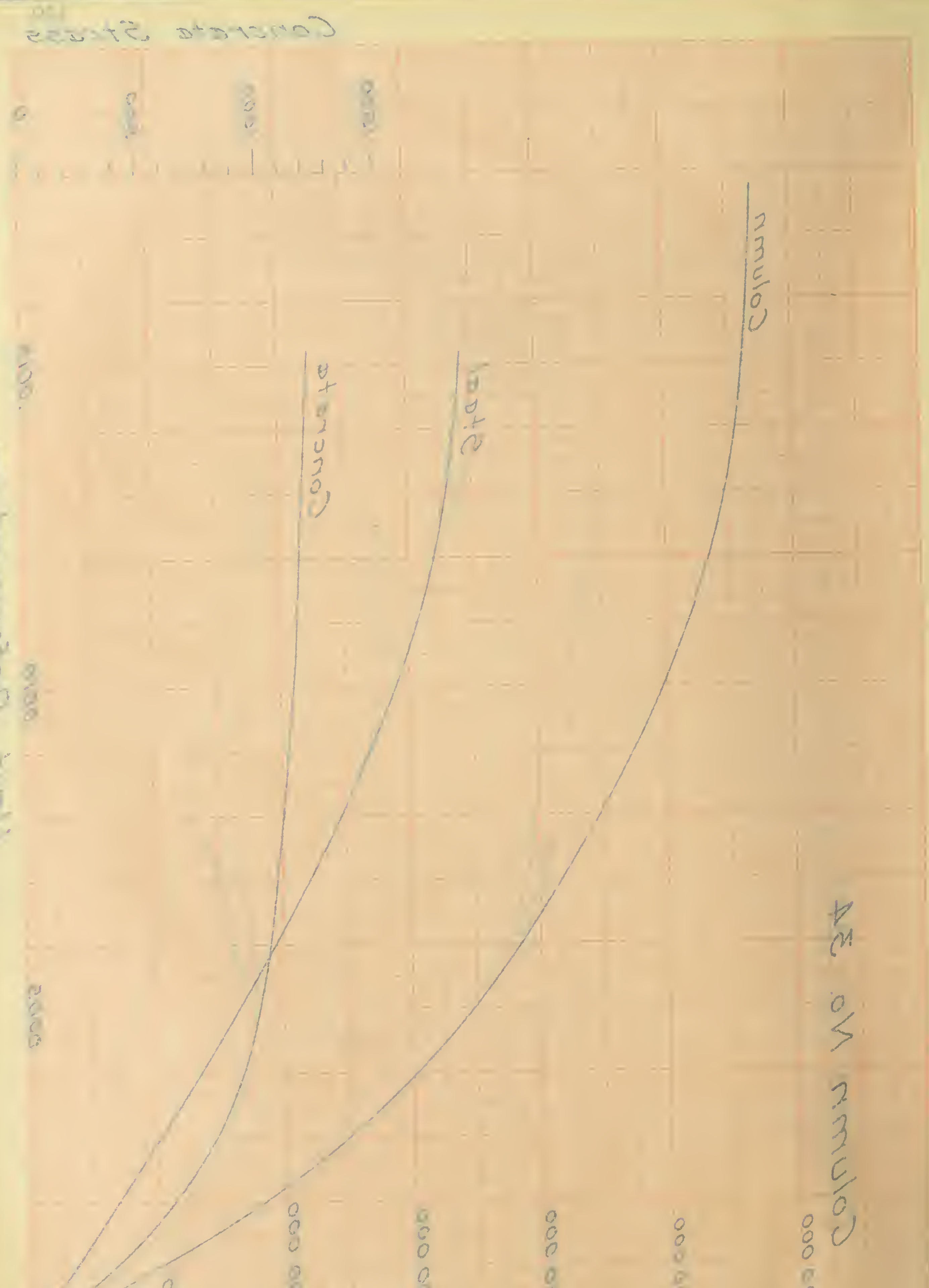
Column No. 31
Concrete Stress
0
500
1000
1500
Unit Deformation
.0005
.0010
.0015
Load in Pounds
100
200000
300000
400000
500000
Column
Concrete
Steel

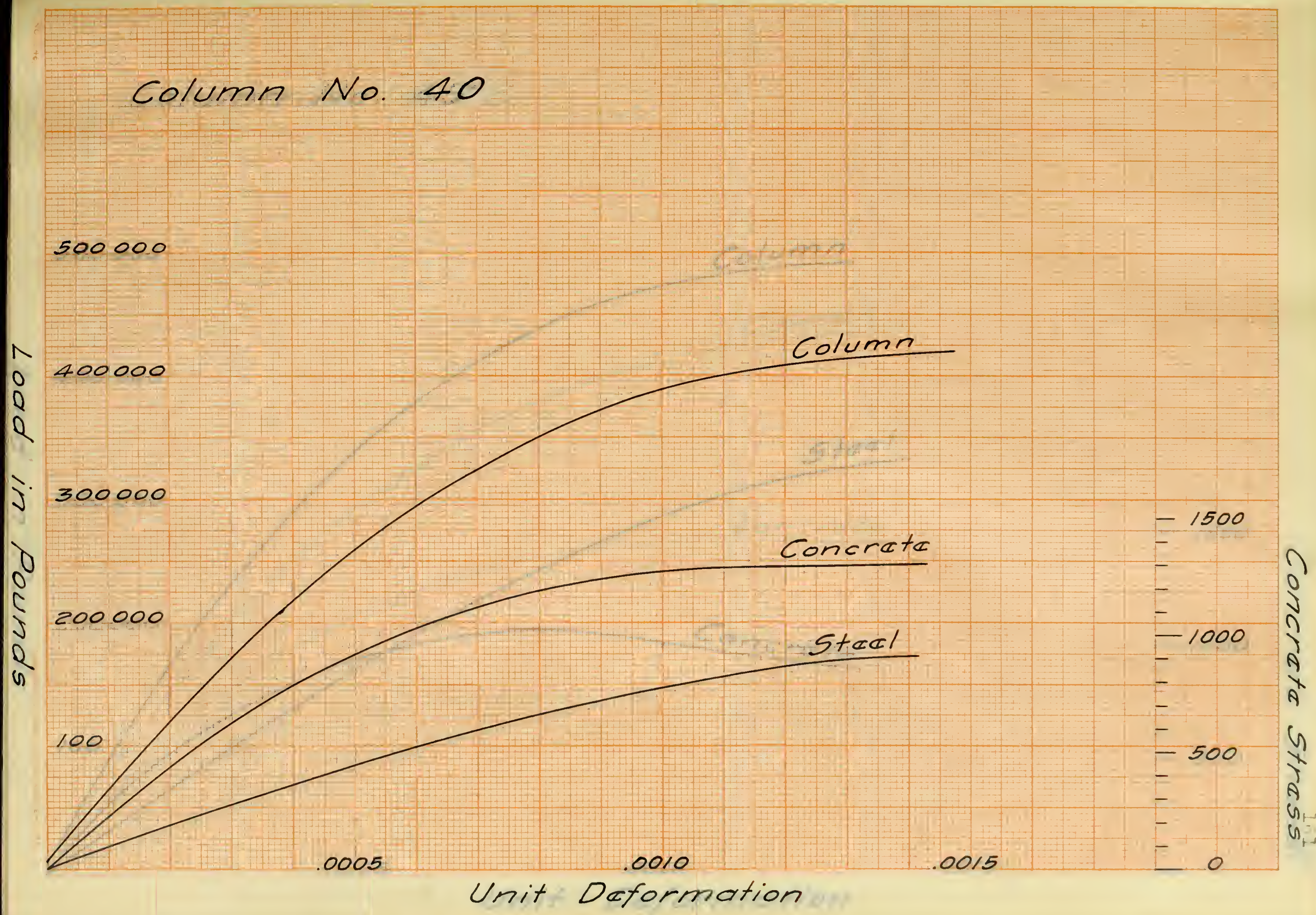
Column No. 32
400 000
300 000
200 000
100
Load in Pounds
Column
Steel
Concrete
.0005
.0010
.0015
Unit Deformation
1500
1000
500
0
Concrete Stress



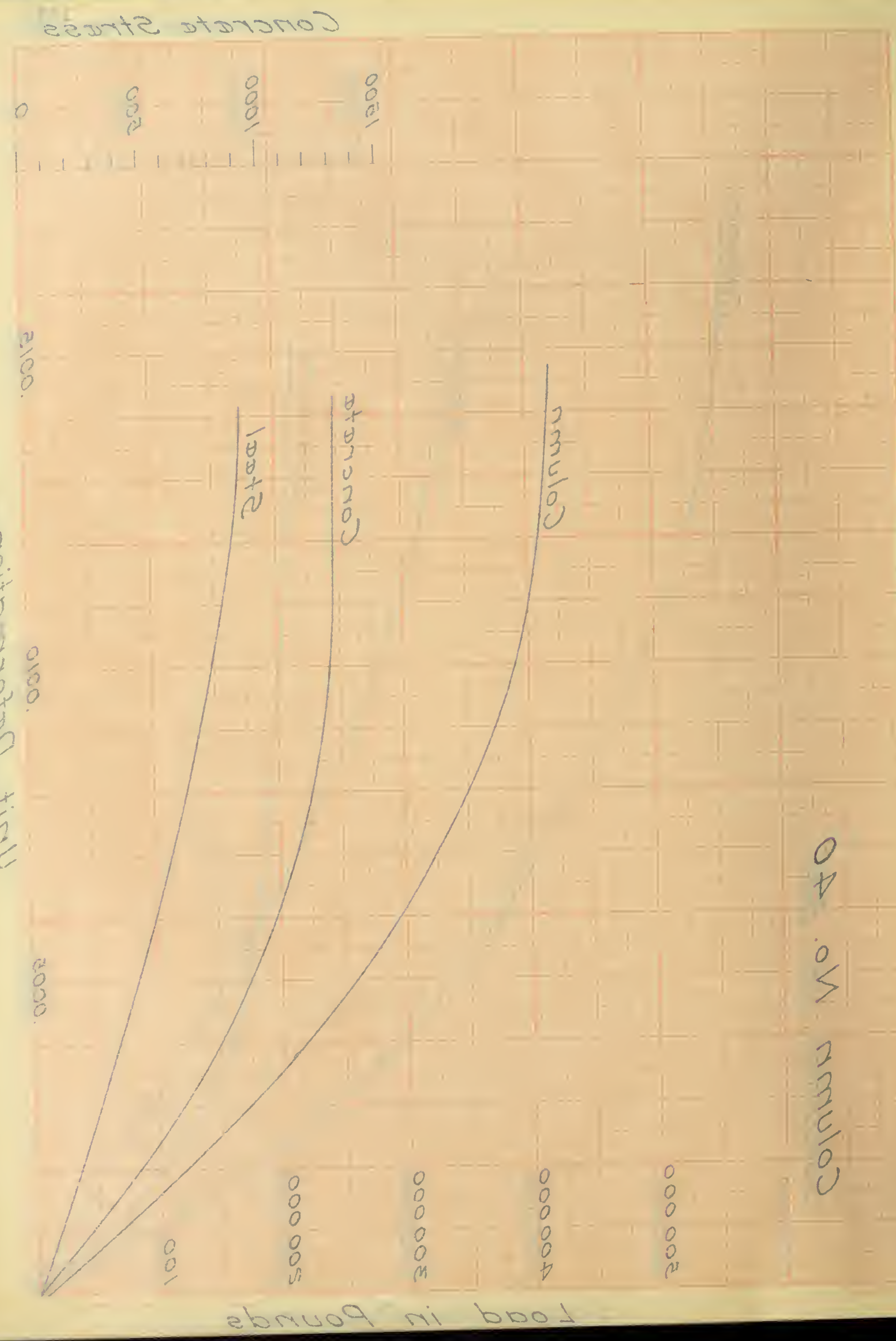
Concrete Stress
0
500
1000
1500
Unit Deformation
.0005
.0010
.0015
Load in Pounds
100 000
200 000
300 000
400 000
Steel
Concrete
Column
Column No. 32

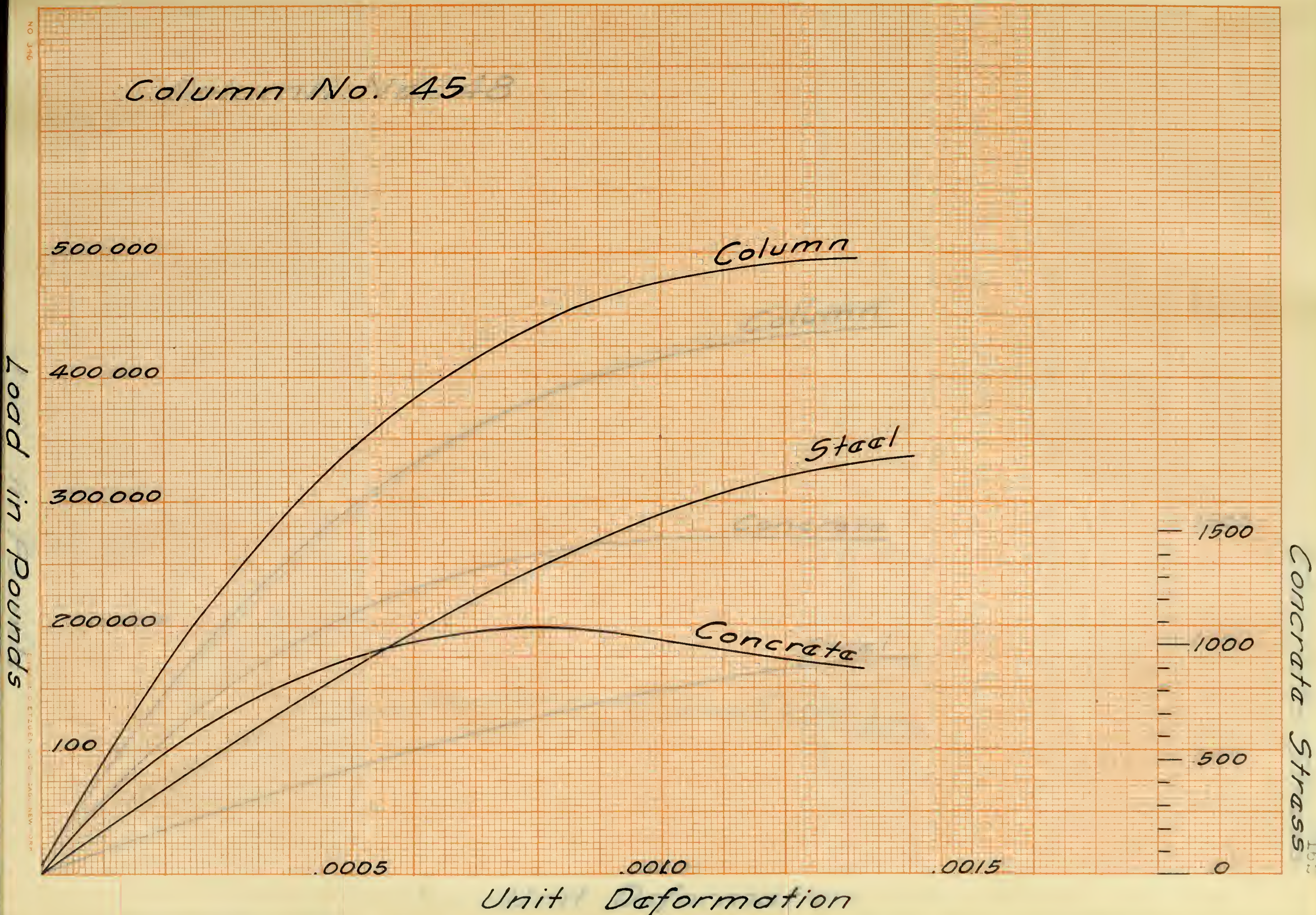

Column No. 34
600 000
500 000
400 000
300 000
200 000
100
Column
Steel
Concrete
.0005
.0010
.0015
1500
1000
500
0
Load in Pounds
Unit Deformation
Concrete Stress

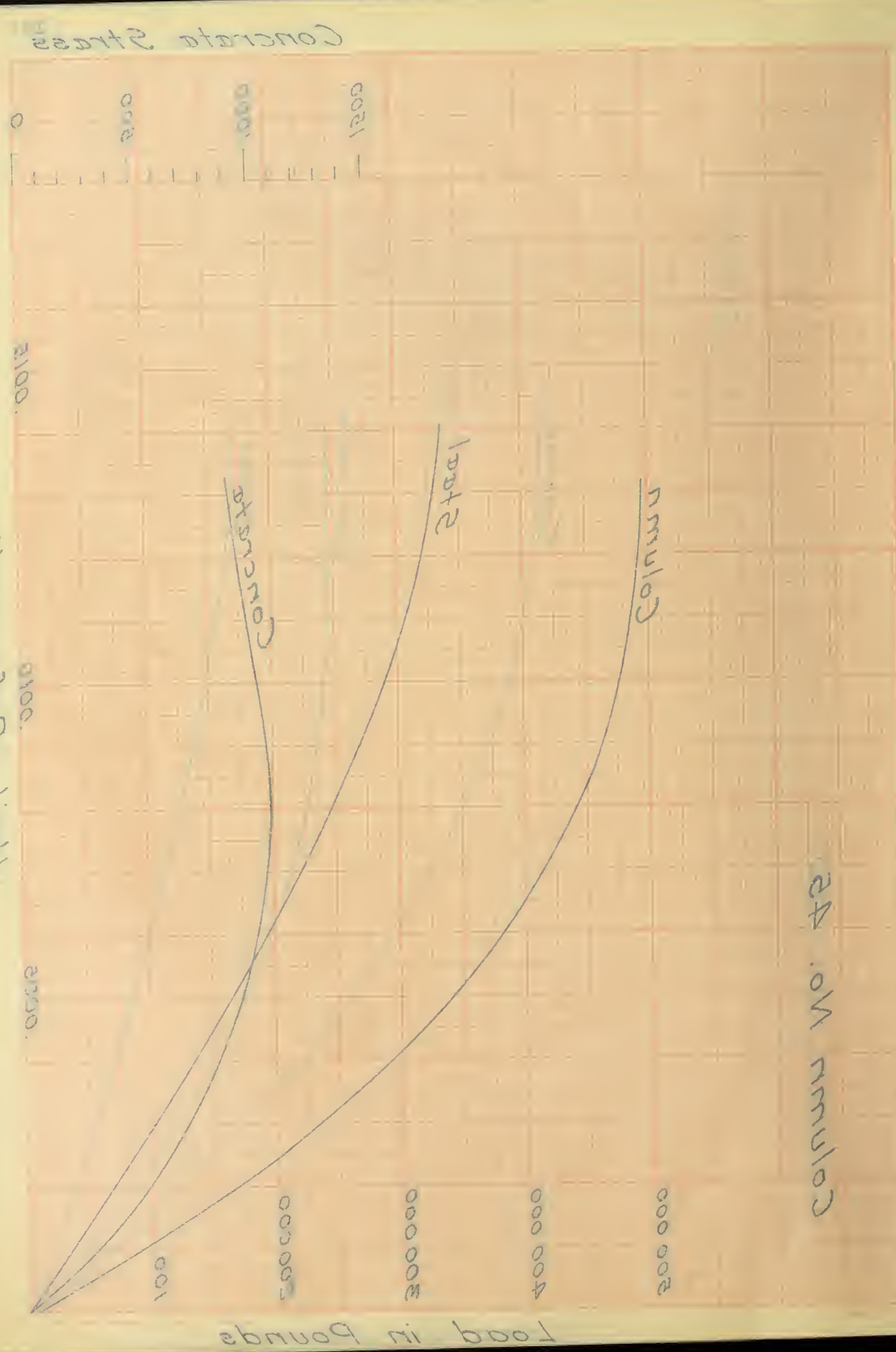

Column No. 34
Load in Pounds
Unit Deformation
Concrete Stress
Column
Steel
Concrete



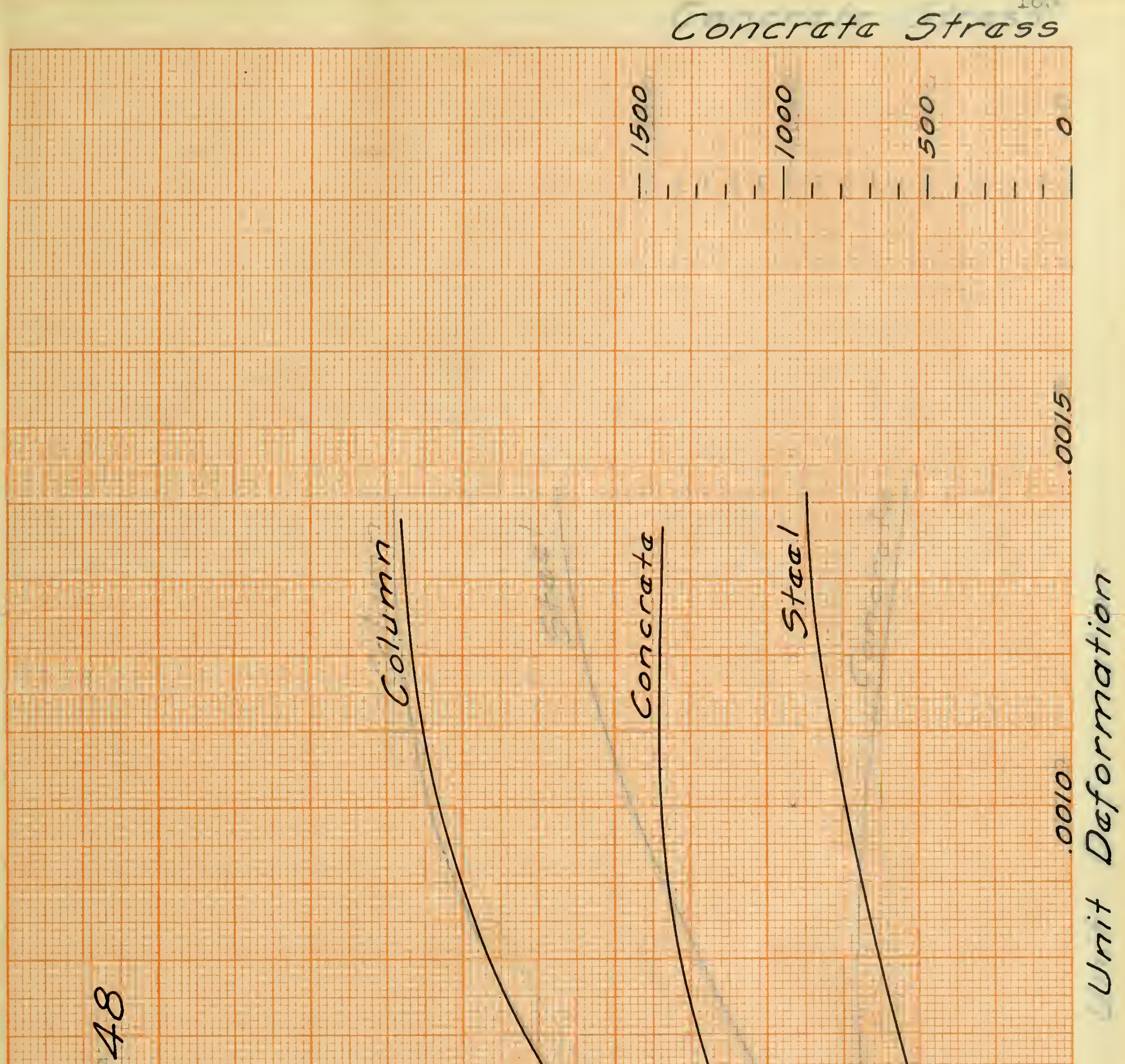
Column No. 40
500 000
400 000
300 000
200 000
100
Load in Pounds
Column
Concrete
Steel
.0005
.0010
.0015
Unit Deformation
1500
1000
500
0
Concrete Stress

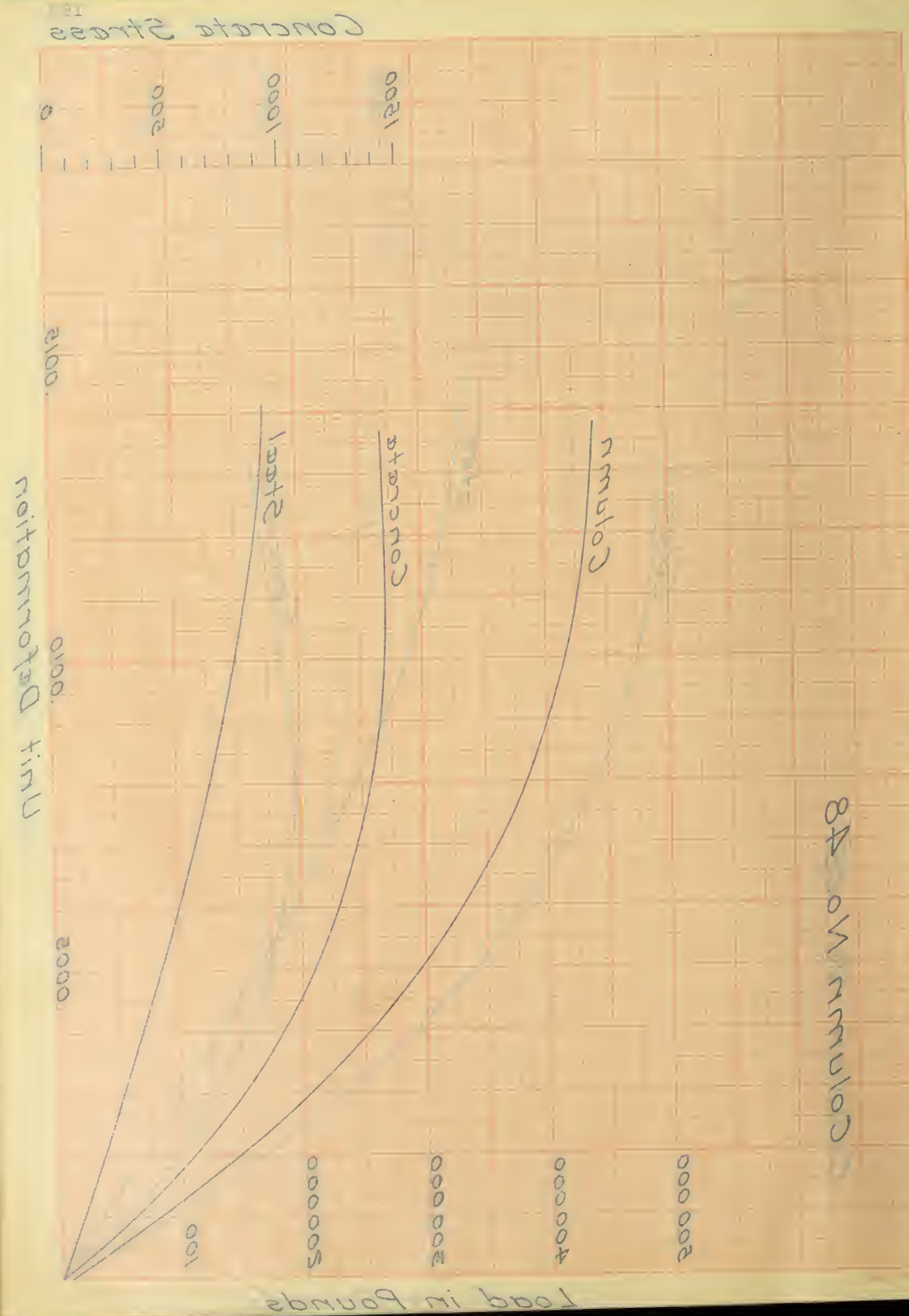

Column No. 40
Concrete Stress
0
500
1000
1500
Unit Deformation
.0005
.0010
.0015
Load in Pounds
100 000
200 000
300 000
400 000
500 000
Column
Concrete
Steel

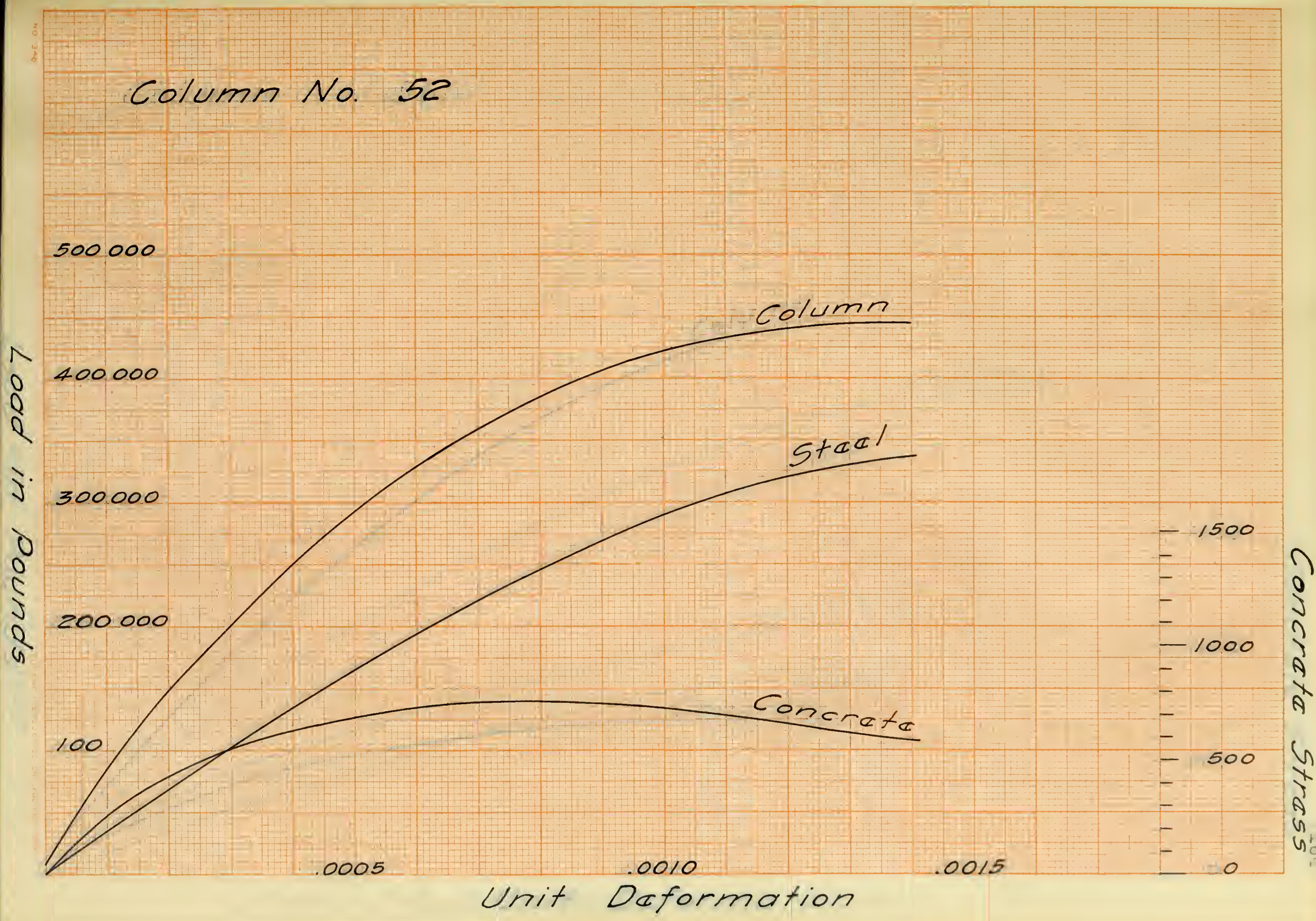

Column No. 45
Load in Pounds
500 000
400 000
300 000
200 000
100
Column
Steel
Concrete
.0005
.0010
.0015
Unit Deformation
Concrete Stress
1500
1000
500
0

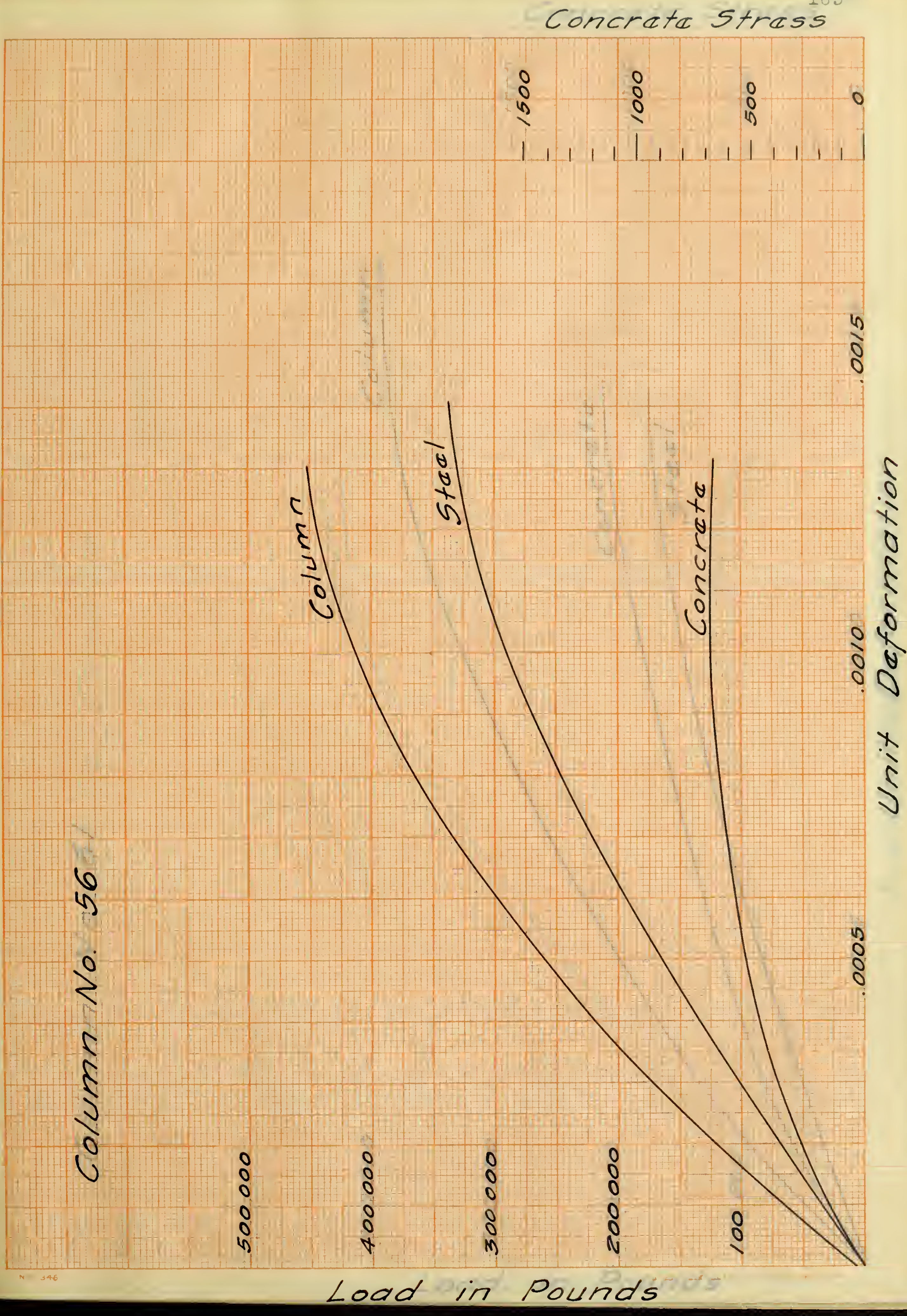


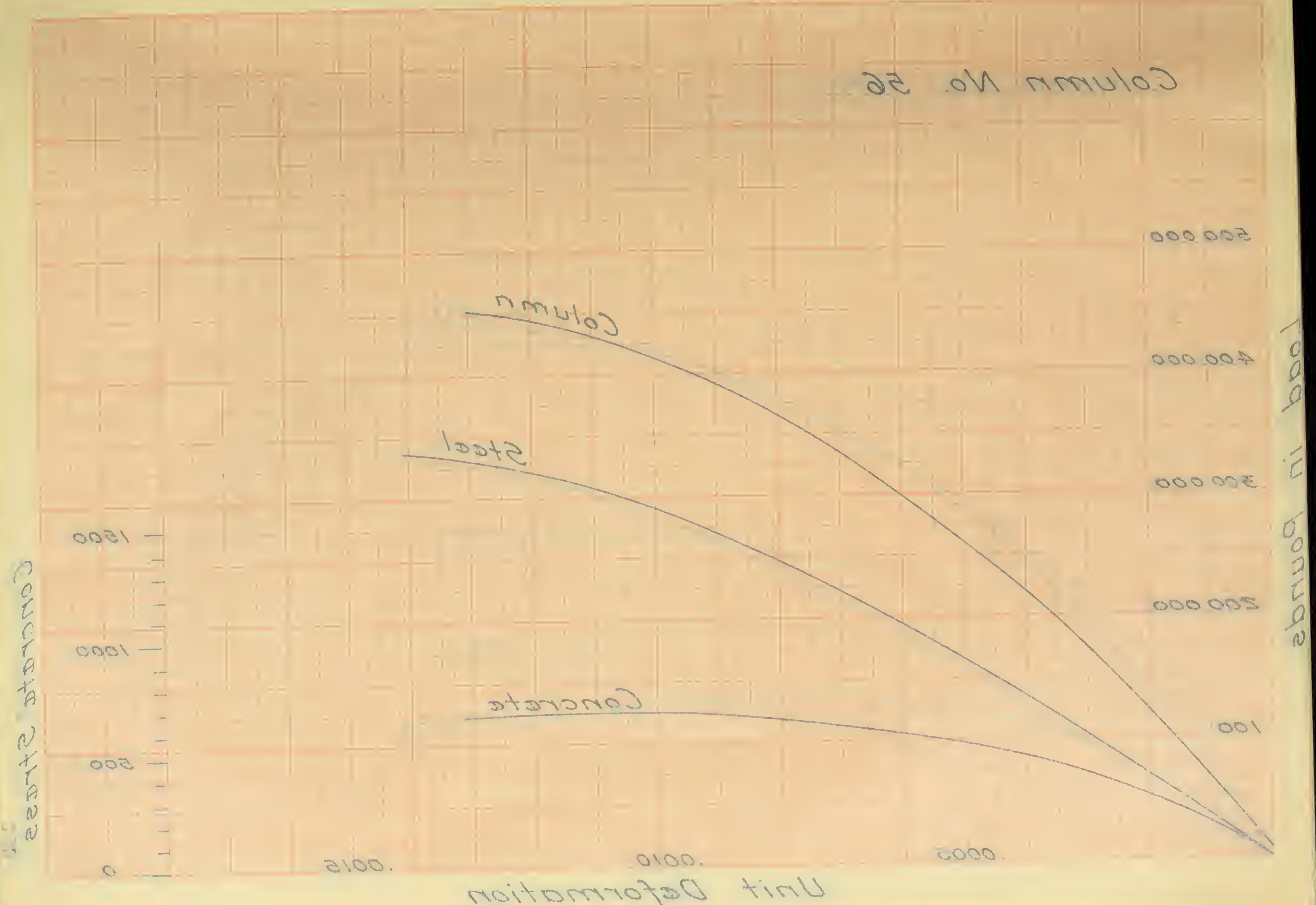
Column No. 45
Concrete Stress
0
500
1000
1500
Unit Deformation
.0015
.0010
.0005
Load in Pounds
100
200000
300000
400 000
500 000
Column
Steel
Concrete


Concrete Stress
1500
1000
500
0
Column No. 48
Column
Concrete
Steel
.0015
.0010
.0005
Unit Deformation
500000
400000
300000
200000
100
Load in Pounds

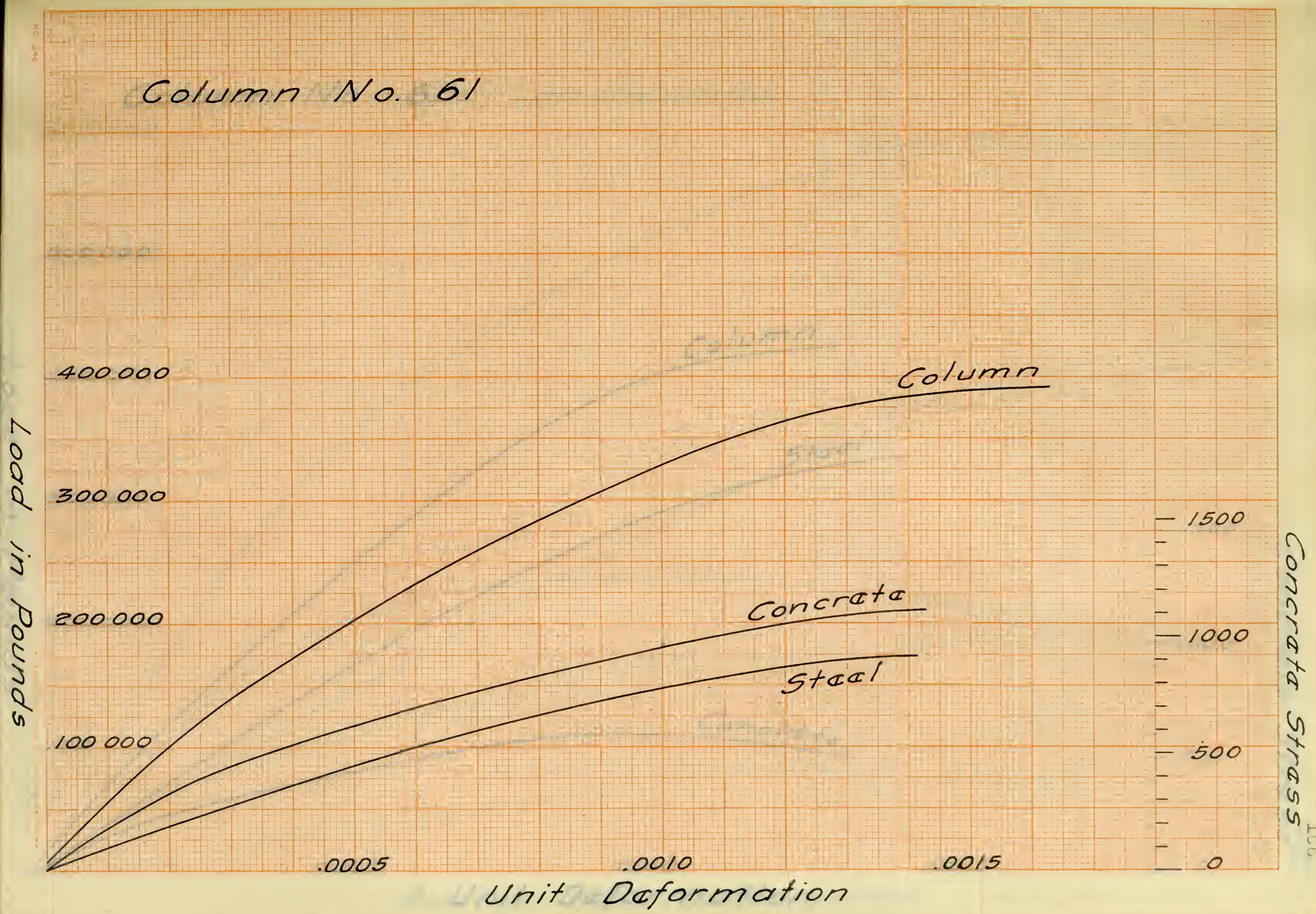

Concrete Stress
0
500
1000
1500
.0015
.0010
.0005
Unit Deformation
Steel
Concrete
Column
Column No. 48
100
200000
300000
400000
500000
Load in Pounds


Column No. 52
Load in Pounds
500.000
400.000
300.000
200.000
100
Column
Steel
Concrete
.0005
.0010
.0015
1500
1000
500
0
Concrete Stress
Unit Deformation

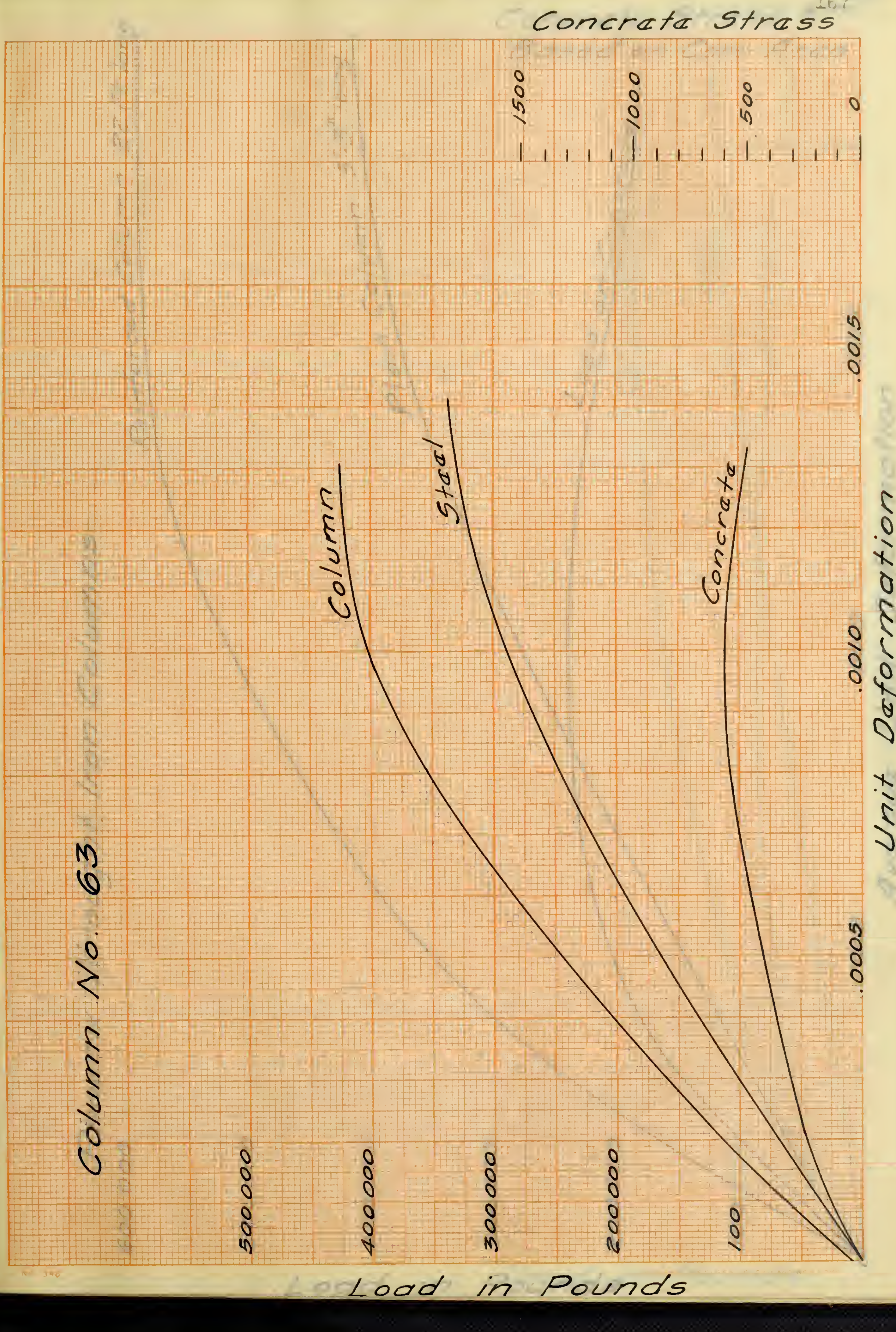

Concrete Stress
1500
1000
500
0
Column No. 56
Column
Steel
Concrete
500 000
400 000
300 000
200 000
100
.0015
.0010
.0005
Unit Deformation
Load in Pounds

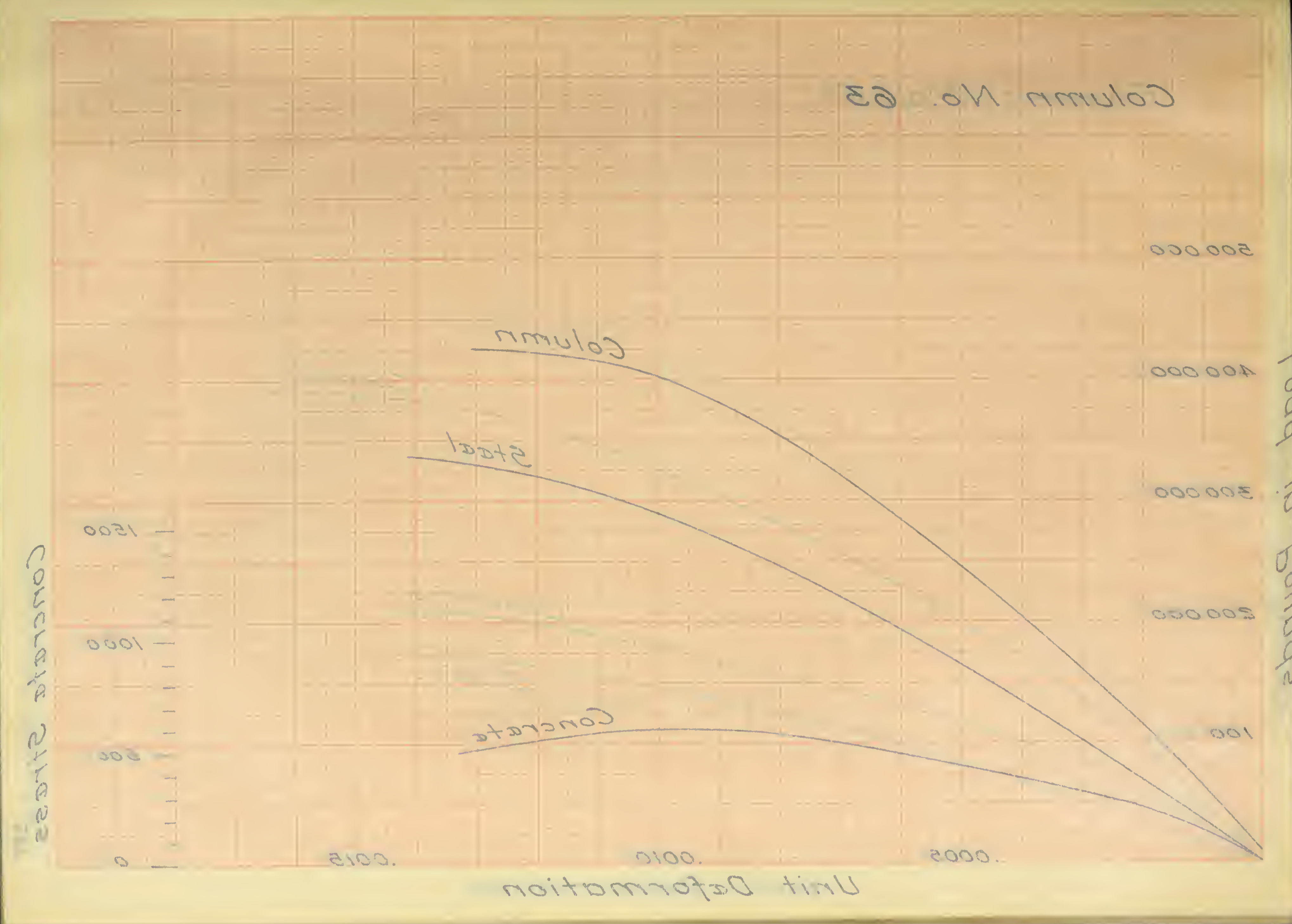


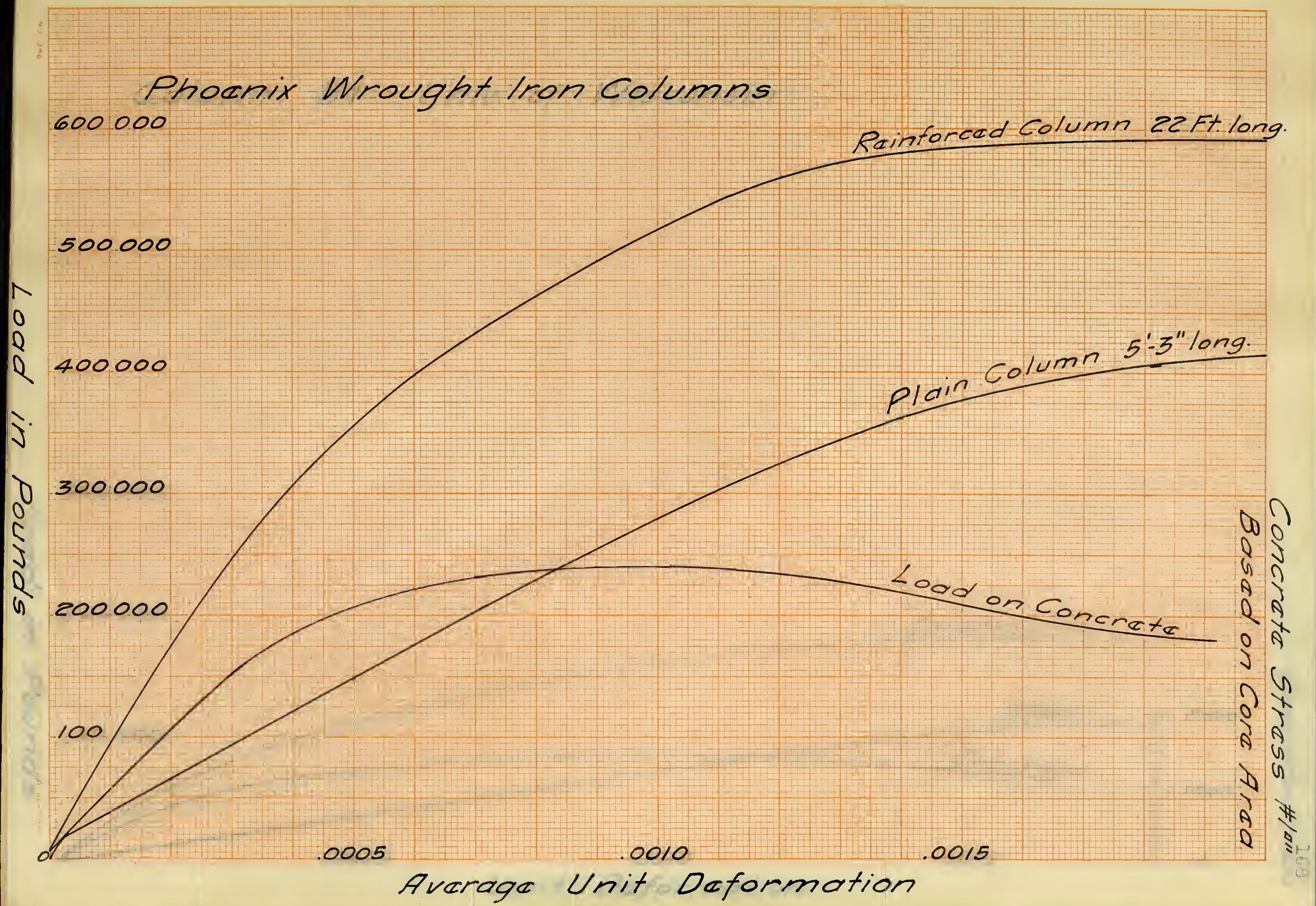
Column No. 56
Column
Steel
Concrete
Load in Pounds
100
200 000
300 000
400 000
500 000
Concrete Stress
0
500
1000
1500
Unit Deformation
.0005
.0010
.0015

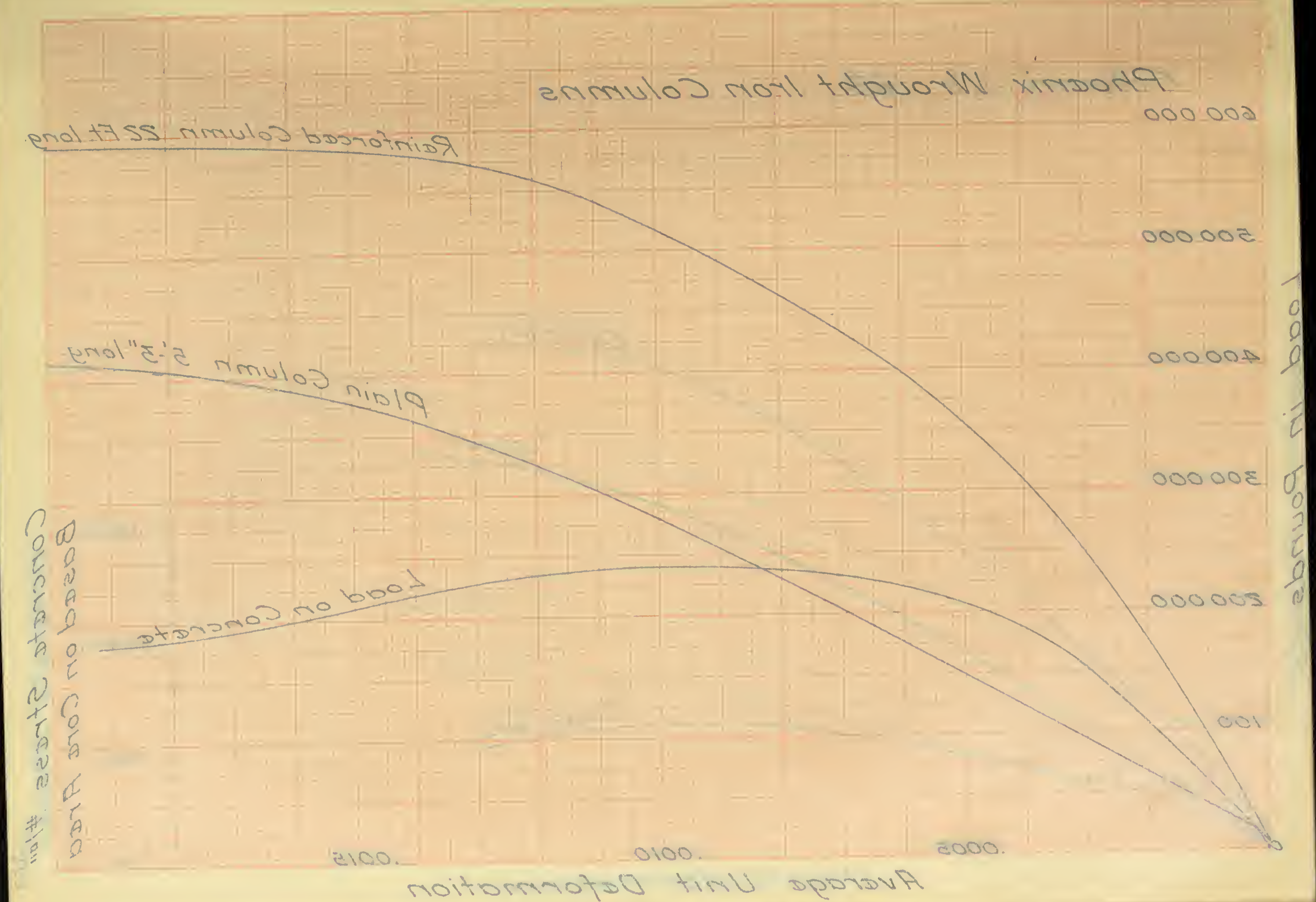


Column No. 61
Load in Pounds
400 000
300 000
200 000
100 000
Column
Concrete
Steel
Concrete Stress
1500
1000
500
0
.0005
.0010
.0015
Unit Deformation

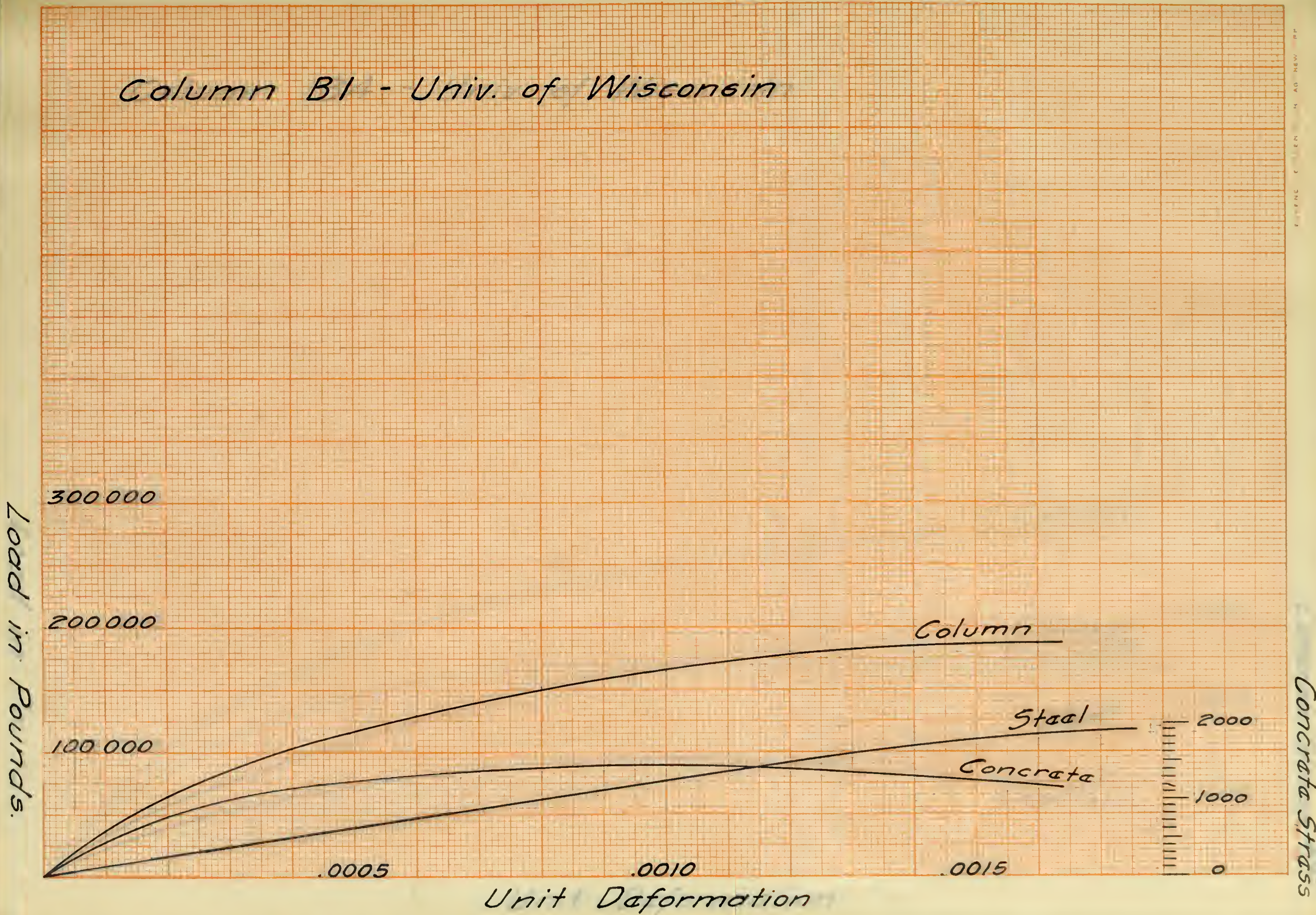

Concrete Stress
1500
1000
500
0
Column No. 63
Column
Steel
Concrete
.0015
.0010
.0005
Unit Deformation
500000
400000
300000
200000
100
Load in Pounds

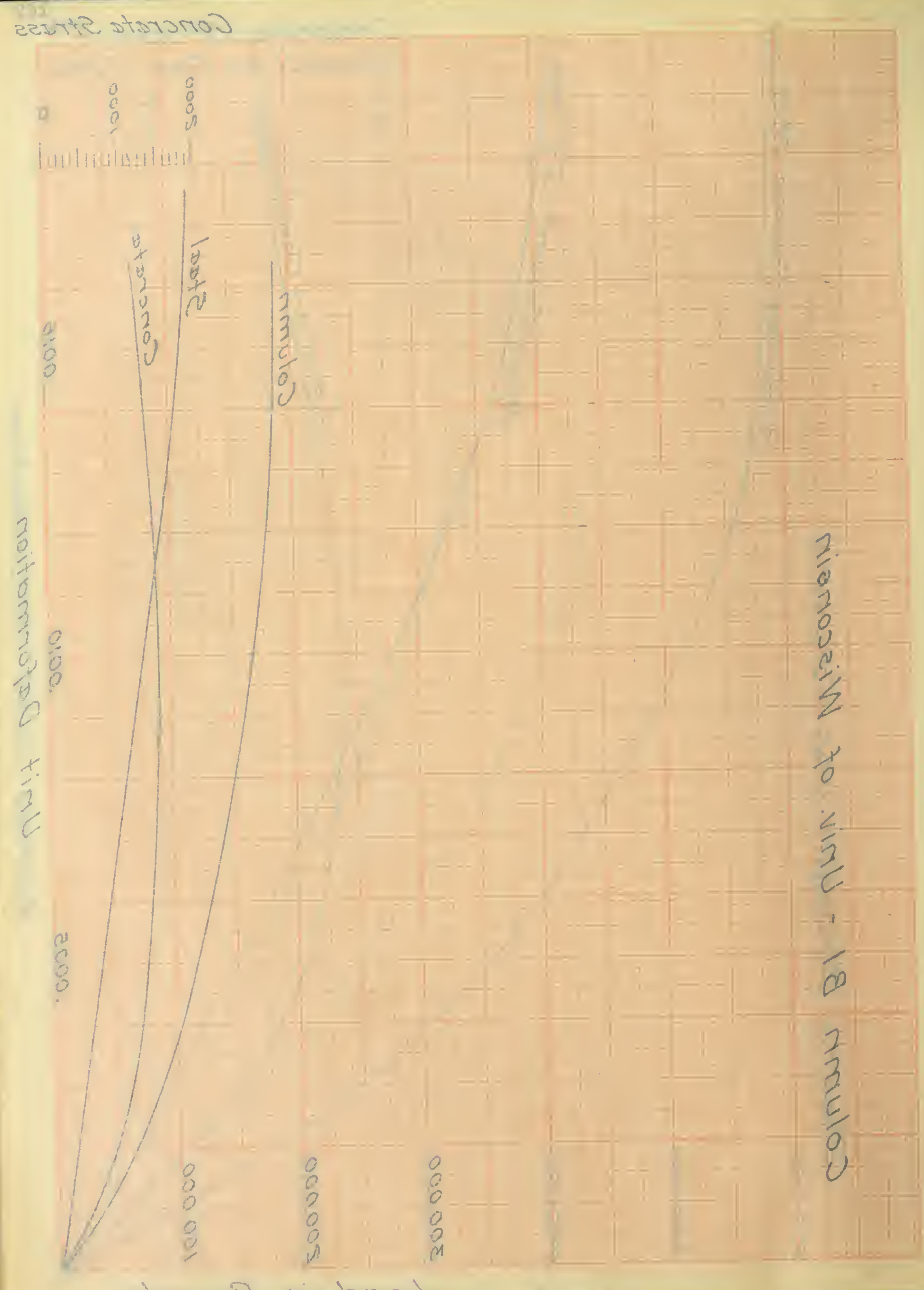

Column No. 63
Column
Steel
Concrete
Concrete Stress
0
500
1000
1500
Unit Deformation
.0005
.0010
.0015
Load in Pounds
100
200 000
300 000
400 000
500 000

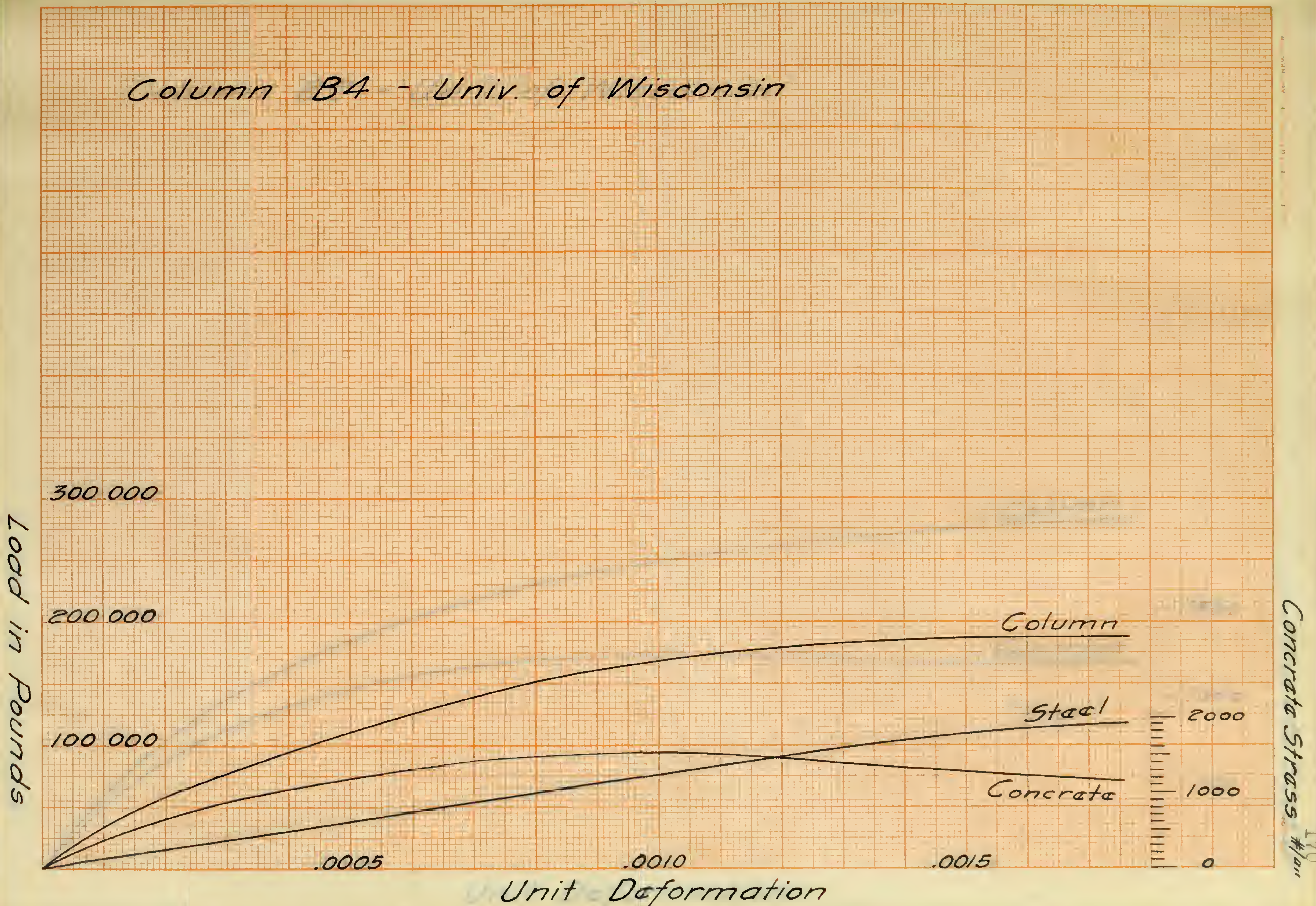

Phoenix Wrought Iron Columns
Reinforced Column 22 Ft. long.
Plain Column 5'-3" long.
Load on Concrete
Concrete Stress #/□"
Based on Core Area
600.000
500.000
400.000
300.000
200.000
100
0
.0005
.0010
.0015
Load in Pounds
Average Unit Deformation

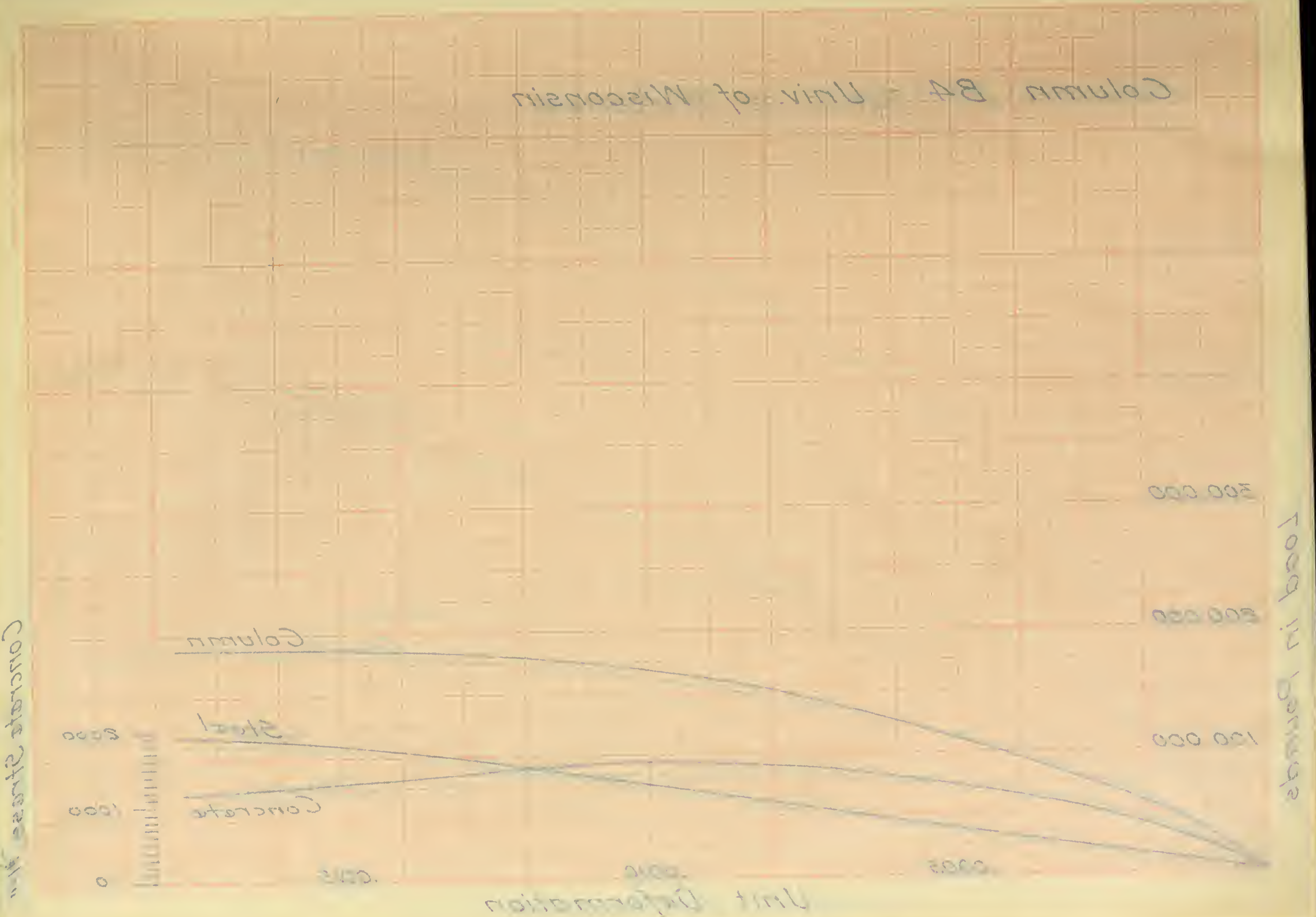


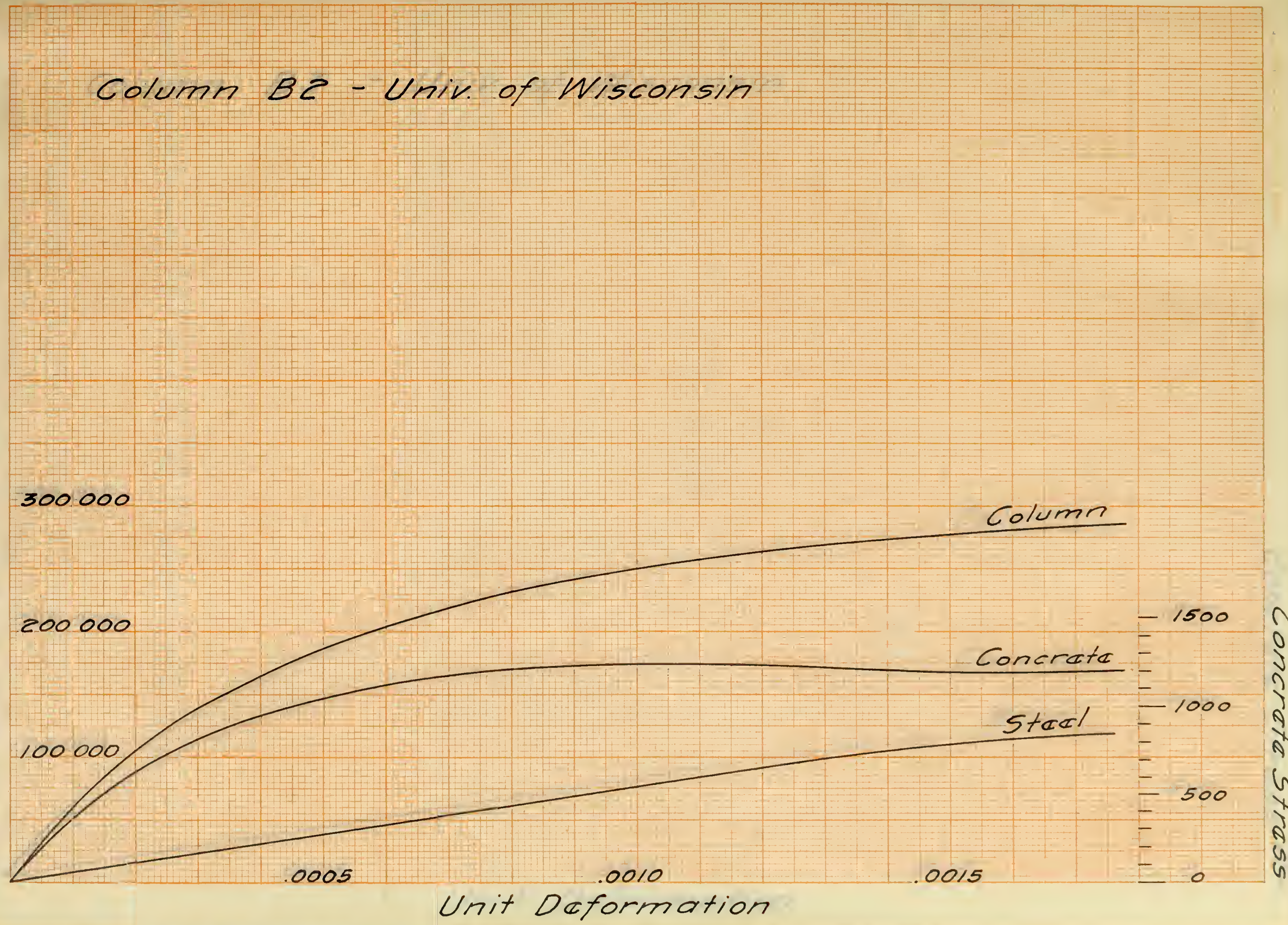
Phoenix Wrought Iron Columns
Reinforced Column 22 Ft long
Plain Column 5'-3" long
Load on Concrete
600 000
500 000
400 000
300 000
200 000
100
Load in Pounds
.0005
.0010
.0015
Average Unit Deformation
Concrete Stress #/□"
Based on Core Area


Column B1 - Univ. of Wisconsin
300 000
200 000
100 000
Load in Pounds.
Column
Steel
Concrete
2000
1000
0
Concrete Stress
.0005
.0010
.0015
Unit Deformation

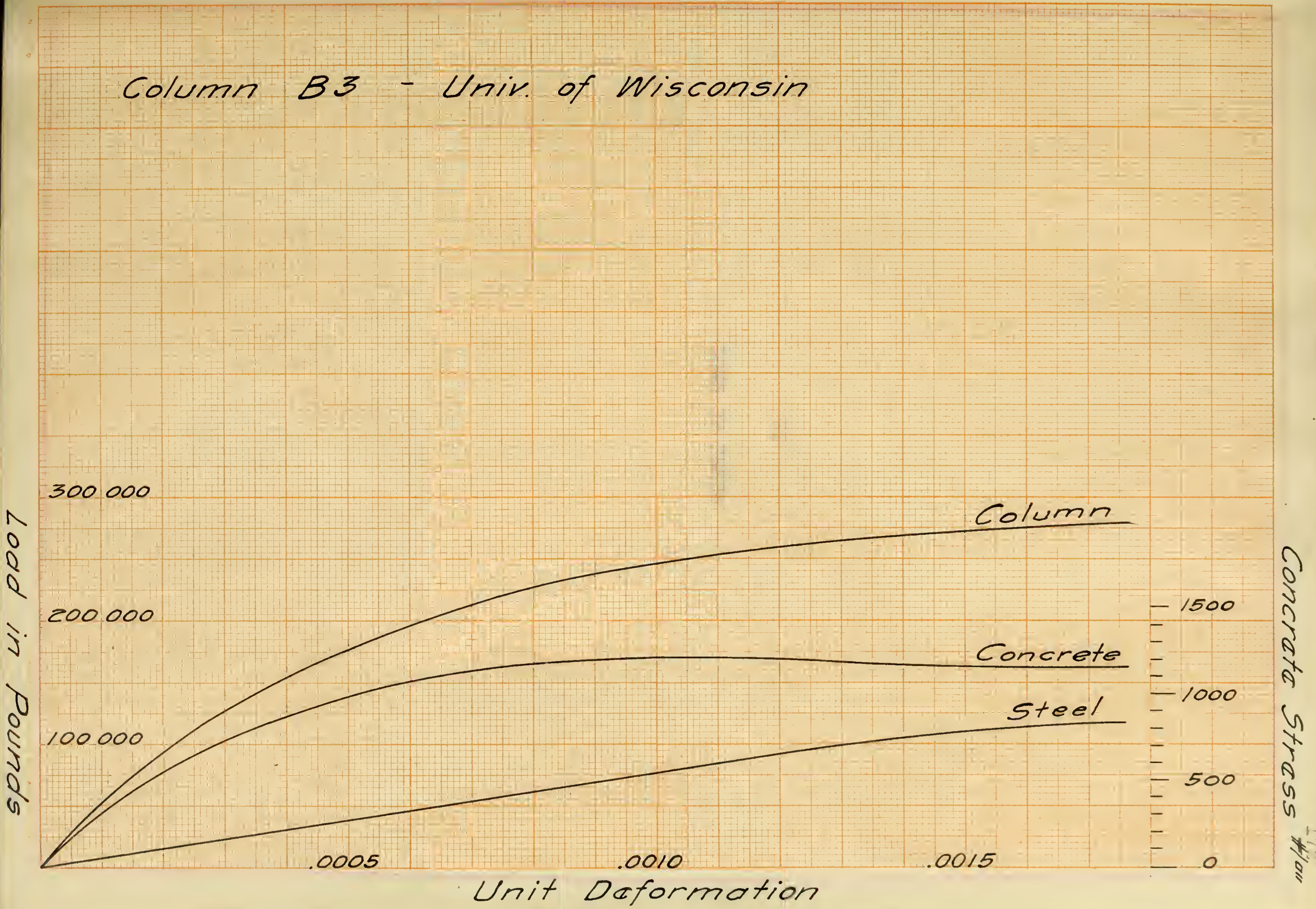

Column B1 - Univ. of Wisconsin
Concrete Stress
0
1000
2000
Steel
Concrete
Column
.0015
.0010
.0005
Unit Deformation
100 000
200000
300000
Load in Pounds.


Column B4 - Univ. of Wisconsin
300 000
200 000
100 000
Load in Pounds
Column
Steel
Concrete
2000
1000
0
Concrete Stress #/□"
.0005
.0010
.0015
Unit Deformation

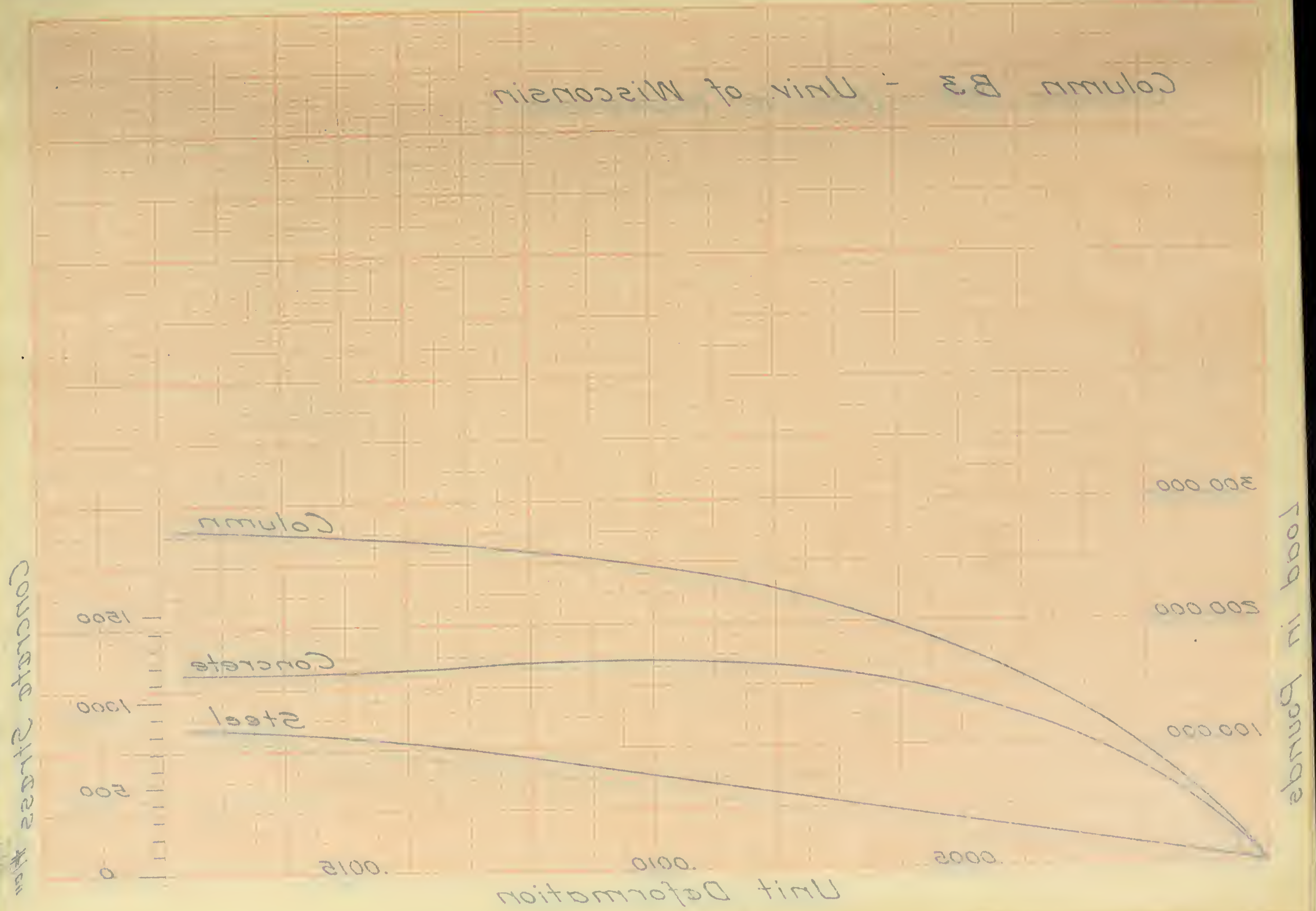




Column B2 - Univ. of Wisconsin
300 000
200 000
100 000
Load in Pounds
.0005
.0010
.0015
Unit Deformation
Column
Concrete
Steel
1500
1000
500
0
Concrete Stress



Column B3 - Univ. of Wisconsin



Column B3 - Univ. of Wisconsin
Column
Concrete
Steel
Concrete Stress #/in²
0
500
1000
1500
.0015
.0010
.0005
Unit Deformation
100 000
200 000
300 000
Load in Pounds

VI

LOGS OF TESTS

PLAIN STEEL COLUMNS

SUMMARY OF AVERAGE UNIT DEFORMATIONS

	Column Number					
Load	8905	8906	8905-6	8910	8911	8910-1
0	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
25 000	61	51	56	78	60	69
50 000	115	120	118	120	124	122
75 000	180	188	184	176	194	185
100 000	242	246	244	266	244	255
125 000	302	308	305	310	310	310
150 000	371	388	379	370	380	375
175 000	434	440	437	442	456	448
200 000	510	498	504	522	539	531
225 000	585	580	582	596	614	605
250 000	664	645	655	678	692	685
275 000	739	722	730	762	776	769
300 000	829	795	812	852	864	858
325 000	926	888	907	938	959	949
350 000	1042	983	1012	1058	1078	1068
375 000	1169	1087	1128	1212	1190	1201
400 000	1340	1224	1282	1536	1386	1461
425 000	1650	1440	1545			
Ultimate Load	440 200	449 000	444 600	410 400	425 600	418 000

PLAIN STEEL COLUMNS

SUMMARY OF AVERAGE UNIT DEFORMATIONS

(Continued)

Load	8915	8916	8915-6	8920	8921	8920-1
	Column Number					
0	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
25 000	56	54	55	56	56	56
50 000	114	110	112	114	111	112
75 000	175	166	170	202	172	187
100 000	244	232	238	244	236	240
125 000	310	300	305	326	304	315
150 000	386	372	379	400	381	390
175 000	467	451	459	480	462	471
200 000	545	528	536	558	548	553
225 000	628	611	620	687	641	664
250 000	718	702	710	753	737	745
275 000	817	793	805	854	842	848
300 000	920	913	917	959	952	955
325 000	1030	1027	1028	1093	1082	1088
350 000	1187	1170	1178	1193		
375 000		1421				
Ultimate Load	368 000	376 000	372 000	374 200	345 000	359 600

REINFORCED STEEL COLUMNS

SUMMARY OF AVERAGE UNIT DEFORMATIONS

Load	Column Number 8907	8908	8912	8913	8917	8918
0	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
25 000	44	..	34	34	46	39
50 000	82	..	70	72	84	77
75 000	128	132	110	118	130	119
100 000	172	167	161	164	180	165
125 000	218	211	210	204	230	214
150 000	254	265	264	248	294	269
175 000	320	314	322	298	354	326
200 000	384	368	380	348	416	378
225 000	441	412	443	404	483	439
250 000	498	456	506	460	548	499
275 000	570	515	546	518	614	569
300 000	641	575	635	576	693	631
325 000	706	633	705	638	768	698
350 000	775	702	778	692	846	760
375 000	844	751	861	760	936	832
400 000	926	834	950	824	1030	903
425 000	1025	888	1045	892	1160	985
450 000	1109	957	1165	966		1068
475 000	1240	1027	1357	1044		1161
500 000	1348	1098	1734	1136		1323
525 000	1535	1182		1234		
550 000	1755	1300		1416		
575 000		1475				
Ultimate Load	577 000	602 000	510 000	584 700	468 500	532 200

REINFORCED STEEL COLUMNS

SUMMARY OF AVERAGE UNIT DEFORMATIONS

(Continued)

	Column Number					
Load	8922	8923	8925	8926	8927	8928
0	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
25 000	35	37	39	36	36	46
50 000	75	77	75	78	82	87
75 000	119	122	111	126	138	143
100 000	175	168	153	164	184	184
125 000	221	218	190	212	243	250
150 000	276	270	226	256	302	308
175 000	338	327	275	302	362	369
200 000	410	368	319	349	426	437
225 000	461	451	370	392	492	496
250 000	541	508	416	446	558	560
275 000	614	575	469	502	627	629
300 000	694	636	523	562	706	696
325 000	776	707	577	604	770	766
350 000	870	772	630	672	848	847
375 000	951	839	679	728	924	923
400 000	1019	911	741	788	1015	1005
425 000		992	796	848	1103	1089
450 000		1082	857	902	1209	1197
475 000		1193	916	964	1350	1306
500 000			979	1028	1604	1549
525 000			1054	1104		
550 000			1132	1174		
575 000			1236	1292		
600 000			1368	1420		
625 000			1632	1654		
650 000				2050		
Ultimate Load	491 400	495 500	636 000	655 000	516 000	530 500

REINFORCED STEEL COLUMNS

SUMMARY OF AVERAGE UNIT DEFORMATIONS

(Continued)

	Column Number		
Load	8929	8930	8931
0	0.000000	0.000000	0.000000
25 000	35	45	32
50 000	61	86	60
75 000	103	127	97
100 000	137	169	128
125 000	175	213	167
150 000	214	257	212
175 000	253	314	255
200 000	291	365	297
225 000	345	410	347
250 000	389	471	391
275 000	435	520	442
300 000	485	574	492
325 000	535	629	547
350 000	585	682	608
375 000	635	746	672
400 000	684	803	738
425 000	738	858	804
450 000	794	934	873
475 000	852	1001	947
500 000	911	1079	1026
525 000	977	1167	1112
550 000	1039	1253	1216
575 000	1139	1368	1344
600 000	1223	1518	1504
625 000		1790	1786
Ultimate Load	600 000+	630 700	635 700

REINFORCED STEEL COLUMNS

SUMMARY OF AVERAGE UNIT DEFORMATIONS

(Continued)

Load	8933	8934	8935	8936	8937	8938
	Column Number					
0	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
25 000	46	38	40	33	38	38
50 000	75	70	68	69	72	66
75 000	121	103	110	110	104	109
100 000	155	144	143	148	142	155
125 000	193	186	185	196	182	195
150 000	242	225	222	238	222	244
175 000	288	272	272	290	270	288
200 000	335	328	323	334	318	338
225 000	388	378	370	386	366	390
250 000	440	433	418	439	414	435
275 000	498	495	470	486	466	491
300 000	549	545	520	546	518	544
325 000	604	617	574	614	572	602
350 000	666	674	626	667	622	651
375 000	711	749	728	727	680	702
400 000	773	821	734	785	730	761
425 000	831	890	788	842	792	819
450 000	891	964	844	908	850	889
475 000	960	1056	895	984	914	951
500 000	1027	1159	954	1055	984	1029
525 000	1110	1268	1026	1138	1044	1116
550 000	1188	1408	1094	1232	1130	1202
575 000	1322	1561	1180	1376	1232	1323
600 000	1513	1818	1390	1595	1402	1522
0	350	712	228	734	308	403

SUMMARY OF LOADS CARRIED
AT VARIOUS UNIT DEFORMATIONS

Unit Deform.	8905	8906	8907	8908	8910	8911	8912	8913	8915	8916
0.0000	0	0	0	0	0	0	0	0	0	0
1	42	42	60	62	42	42	68	65	44	45
2	83	83	115	120	82	82	121	123	84	86
3	122	122	168	170	121	120	165	176	121	126
4	160	161	206	216	158	155	207	224	154	159
0.0005	196	200	250	266	194	190	247	268	186	192
6	229	235	286	311	226	221	286	319	216	221
7	260	269	322	349	256	252	322	351	245	250
8	290	300	360	390	286	281	355	390	271	276
9	318	329	392	428	314	309	386	427	295	299
0.0010	341	355	419	465	338	335	414	461	319	321
11	361	378	443	500	357	357	436	490	337	339
12	379	396	467	529	372	377	455	516	352	353
13	395	410	491	550	384	391	469	535		365
14	405	421	515	565	393	402		548		373
0.0015	414	431	537	579	398					
16	422		557	592						
17	427									
U. Load	440	449	577	602	410	426	510	585	368	376

SUMMARY OF LOADS CARRIED

AT VARIOUS UNIT DEFORMATIONS

(Continued)

Unit Deform.	Column Number 8917	8918	8920	8921	8922	8923	8925	8926	8927	8928
0.0000	0	0	0	0	0	0	0	0	0	0
1	57	63	42	44	64	60	66	60	60	55
2	109	118	80	84	115	114	132	119	107	105
3	154	164	117	123	159	162	189	174	150	146
4	193	208	150	156	199	206	240	226	190	187
0.0005	232	250	182	186	235	246	290	275	228	227
6	268	288	211	214	269	285	336	320	265	264
7	303	326	237	240	302	322	382	363	300	301
8	336	363	262	265	331	358	426	405	334	335
9	366	397	286	288	358	395	467	446	366	388
0.0010	393	429	308	310	387	427	505	486	397	399
11	414	458	329	329		454	539	522	424	427
12	432	482	349			477	568	553	447	452
13		497					589	577	466	471
14		509					604	594	480	486
0.0015							615	609	492	495
16							624	619	500	500
17								627		
18								635		
19								642		
0.0020								647		
U. Load	468	532	365	345	491	495	636	655	516	530

SUMMARY OF LOADS CARRIED

AT VARIOUS UNIT DEFORMATIONS

(Continued)

Unit Deform.	Column Number 8929	8930	8931	8933	8934	8935	8936	8937	8938
0.0000	0	0	0	0	0	0	0	0	0
1	74	59	78	64	70	71	68	70	69
2	141	117	144	126	134	136	129	135	124
3	202	168	201	181	187	192	183	190	179
4	257	218	254	231	235	242	234	242	230
0.0005	308	265	302	277	279	291	280	291	279
6	357	310	347	322	321	338	324	338	326
7	407	356	387	367	358	384	365	383	372
8	452	398	424	409	394	430	406	427	416
9	494	438	459	451	428	475	446	468	456
0.0010	534	474	492	489	459	516	483	508	491
11	566	506	521	522	487	552	515	541	522
12	594	536	546	550	511	578	541	567	543
13		561	567	571	530	593	563	587	570
14		581	585	588	545	600	579	600	581
0.0015		598	599	598	557		592		598
16		611	610		568		600		
17		620	618		578				
18			626		588				
19					598				
U. Load		631	636	Other columns not crushed.					

COLUMN NO. 8 9 0 2

Length: 2' – 0"

Plain Steel

Observed Deformation over 10 Inches

Load	NW	NE	SE	SW
0	0.0000	0.0000	0.0000	0.0000
25	2	4	2	2
50	10	12	10	4
75	20	22	14	8
100	24	22	16	10
125	30	28	24	18
150	36	36	32	22
175	42	42	34	28
200	50	46	42	34
225	56	52	48	42
250	62	56	52	50
275	68	62	58	56
300	76	66	62	60
325	82	72	70	70
350	90	80	78	76
375	100	86	86	86
400	104	96	94	94
425	114	102	102	106
450	136	130	128	120
475	186	202	158	134

487 300 Ultimate Load

Very gradual failure. Machine was not run down, so no visible bending occurred.

NOTE: Load stated in thousands of pounds, and deformation stated in inches.

COLUMN NO. 8 9 0 5

Length: 4' - 8"

Plain Steel.

Observed Deformation over 41 Inches

Load	NW	NE	SE	SW
0	0.0000	0.0000	0.0000	0.0000
25	16	18	16	48
50	22	40	42	84
75	40	60	68	128
100	72	78	102	148
125	182	198	132	274
150	204	220	168	296
175	232	248	194	318
200	260	274	228	354
225	292	298	264	386
250	326	330	288	416
275	358	358	328	450
300	394	394	364	494
325	438	436	392	534
350	484	482	446	578
375	542	536	482	636
400	604	614	542	718
425	708	742	676	862

440 200 Ultimate Load

Final failure by bending to North and East. Flanges buckled in lower panel, crimping being more marked on south side.

NOTE

In this and the following logs of test the loads will be stated in thousands of pounds, and the deformation will be given in inches. The average unit deformation is given in another series of tables.

COLUMN NO. 8 9 0 6

Length: 4' – 8"   Plain Steel.

Observed Deformation over 40 Inches

Load	NW	NE	SE	SW
0	0.0000	0.0000	0.0000	0.0000
25	30	20	16	16
50	62	56	42	32
75	90	90	64	56
100	114	114	90	76
125	132	144	116	100
150	156	172	140	122
175	184	194	166	152
200	212	222	186	176
225	242	256	220	212
250	262	290	246	236
275	292	322	276	264
300	322	354	294	302
325	356	392	328	344
350	394	442	360	376
375	438	490	394	416
400	500	560	430	466
425	602	676	468	558

449 000 Ultimate Load.

A very symmetrical failure, bending to South and East. Axis of bend practically at the center of the column length.

COLUMN NO. 8 9 0 7

Length: 4' – 8" — Core Section

Mixture: 1 : 2 : 4 — Age: 60 Days

Observed Deformation over 39 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	18	10	14	18	18	18	22	18
50	30	26	32	38	36	32	32	30
75	42	44	50	58	54	54	54	44
100	52	60	66	82	72	74	68	60
125	64	80	78	102	90	92	94	82
150	74	102	100	130	114	112	118	106
175	90	120	112	150	124	136	138	132
200	100	144	128	182	152	164	166	158
225	116	168	150	206	170	192	190	180
250	136	192	168	234	194	212	218	208
275	152	218	192	252	222	244	246	232
300	176	248	214	302	244	286	268	262
325	196	274	238	336	268	308	300	284
350	216	300	262	364	298	338	326	316
375	236	328	290	398	318	372	350	342
400	258	356	316	436	356	410	384	372
425	290	398	354	476	382	446	420	404
450	316	432	380	518	418	492	462	438
475	354		434	578	462	548	508	484
500	396	520	478	618	502	602	568	532
525	454	594	540	698	564	682	648	598
550	536	722	678	842	698	826	782	700

577 000 Ultimate Load

Failure by general crushing of upper half of column. Cracks first appeared along edges of flanges, followed by general crushing between flanges. Load of 570 000 to 577 000 pounds held for several minutes.

NOTE: N,E,S, and W are reading on the concrete faces.

COLUMN NO. 8 9 C 8

Length: 4' – 8"　　　　Core Section.

Mixture: 1 : 2 : 4　　　　Age: 59 Days

Observed Deformation over 40 3/4 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25								
50								
75	62	84	56	54	38	40	44	48
100	80	90	70	72	54	52	56	62
125	102	114	92	94	72	74	72	78
150	118	130	116	116	96	94	90	104
175	138	158	142	138	106	114	106	124
200	156	182	162	158	130	134	128	146
225	174	196	180	184	152	158	140	164
250	190	214	194	198	166	176	156	188
275	204	244	218	222	186	198	180	216
300	228	266	242	254	216	226	200	244
325	248	294	262	280	240	256	220	272
350	272	320	288	302	264	284	250	300
375	288	336	304	322	288	310	272	334
400	310	372	334	356	318	348	304	370
425	336	400	350	378	342	378	320	396
450	358	424	376	402	370	402	350	438
475	380	452	396	432	394	440	382	466
500	404	478	422	462	424	480	416	506
525	434	520	446	494	464	520	450	542
550	482	550	466	520	516	580	512	614
575	554	626	488	576	634	650	600	708

602 000 Ultimate Load.

Failure by general crushing over entire column.

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 1 0

Length: 10' - 0" Plain Steel

Observed Deformation

Load	100" Gauge Length over Entire Column NW	NE	SE	SW	30" Gauge Length at Center of Col. NW	NE	SE	SW	30" G.L. at Top of Col. NW	SE
0	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
25	66	124	14	110	30	42	14	32	16	10
50	104	180	44	152	46	52	18	40	26	18
75	156	242	92	214	60	74	38	60	42	38
100	222	302	162	276	72	92	54	80	62	52
125	296	374	224	348	98	116	72	102	80	78
150	354	428	290	408	118	132	94	118	96	94
175	422	506	360	478	132	152	110	140	112	110
200	498	584	448	560	156	176	140	164	132	144
225	566	660	526	632	180	198	162	188	156	166
250	642	740	614	718	204	222	184	214	176	194
275	722	824	702	798	228	246	214	238	202	220
300	804	914	802	886	250	270	244	268	222	246
325	888	1004	890	972	274	292	276	294	244	274
350	1004	1126	1024	1078	304	322	314	330	270	310
375	1146	1262	1206	1236	344	354	358	386	316	386
400	1408	1520	1626	1592	410	402	556	500	362	486

410 400 Ultimate Load

Gradual failure. Column bent to North also twisting. When head of machine was run down crimping occurred locally in center panel and panel just below.

COLUMN NO. 8 9 1 1

Length: 10' – 0" Plain Steel

Observed Deformation

	100" Gauge Length over Entire Column				30" Gauge Length at Center of Col.				30" G.L. Bottom.	
Load	NW	NE	SE	SW	NW	NE	SE	SW	NE	SW
0	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
25	66	48	50	74	20	18	22	24	18	24
50	138	110	102	144	42	38	38	44	34	48
75	210	190	166	208	62	62	58	66	54	70
100	276	222	212	266	82	78	76	84	68	90
125	348	284	276	334	106	102	100	102	86	110
150	430	354	342	396	128	120	120	124	114	132
175	514	422	414	476	150	142	140	144	134	152
200	592	508	502	554	170	170	162	168	154	180
225	672	578	578	626	192	192	188	194	176	200
250	754	654	660	698	218	214	212	220	200	228
275	840	736	752	778	240	238	236	244	222	254
300	932	822	840	864	266	260	262	278	250	280
325	1018	916	938	964	292	288	290	306	276	312
350	1138	1034	1052	1088	324	322	328	344	308	352
375	1248	1150	1154	1206	356	357	358	382	338	392
400	1452	1334	1334	1426	400	410	422	446	414	452

425 600 Ultimate Load

Column bent slightly to north of west. Crimping of both west flanges at points about twenty inches from top.

COLUMN NO. 8 9 1 2

Length: 10' – 0" Core Section.

Mixture: 1 : 2 : 4 Age: 60 Days

Observed Deformation over 100 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	76	46	0	2	2	20	54	74
50	126	82	2	26	26	68	102	124
75	168	114	32	58	66	110	156	176
100	216	160	66	110	118	166	216	234
125	266	216	110	156	166	214	266	286
150	318	266	160	212	222	272	322	344
175	378	318	204	274	280	334	384	406
200	432	374	248	334	342	402	442	466
225	492	426	324	396	400	446	510	528
250	552	488	376	458	468	540	574	590
275	614	550	440	528	532	612	640	656
300	672	610	492	594	602	692	702	718
325	748	678	544	670	672	772	772	786
350	808	752	612	752	756	842	842	862
375	888	826	702	834	836	940	922	938
400	962	916	808	934	928	1024	1010	1024
425	1050	1006	904	1038	1028	1126	1096	1112
450	1158	1126	1024	1172	1154	1260	1214	1216
475	1354	1316	1210	1354	1332	1470	1408	1414
500	1760	1614	1620	1848	1706	1870	1820	1834

510 000 Ultimate Load

Concrete crushed at two levels, just above upper edge of 4th and 5th tie plates. Signs of crushing on three faces – N, W, and E.

NOTE: N, E, S, and W readings are taken on concrete faces.

COLUMN NO. 8 9 1 3

Length: 10' – 0" Core Section.

Mixture: 1 : 2 : 4 Age: 62 Days

Observed Deformation over 100 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	38	24	58	48	28	20	22	32
50	76	96	106	94	64	48	76	70
75	114	132	142	132	110	94	112	104
100	174	178	194	176	160	136	154	148
125	212	208	238	222	200	166	192	190
150	262	248	280	266	242	206	232	240
175	314	308	342	322	292	250	276	286
200	368	370	382	372	336	300	324	338
225	422	424	438	434	396	344	372	398
250	474	480	498	490	448	402	430	452
275	534	544	556	552	508	458	482	508
300	590	606	618	612	568	522	536	564
325	648	672	674	676	634	578	594	628
350	702	726	726	728	686	636	648	686
375	768	808	800	800	750	700	714	746
400	834	868	858	860	810	766	778	822
425	892	946	924	940	880	828	840	892
450	970	1018	990	1004	950	912	916	970
475	1046	1094	1062	1086	1020	984	994	1058
500	1130	1190	1140	1172	1104	1068	1076	1142
525	1244	1288	1232	1262	1206	1184	1188	1262
550	1418	1510	1438	1476	1402	1350	1296	1446

584 700 Ultimate Load

Failed with sharp bend in first panel above bottom.

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 1 4

Length: 10' – 0"

Plain Steel

Observed Deformation over 100 Inches

	N. W. Flange				N. E. Flange			
Load	1	2	3	Av.	1	2	3	Av.
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
26	60	54	66	60	88	88	104	94
50	126	112	128	122	142	162	162	156
75	194	170	188	184	200	226	270	232
100	260	232	254	248	258	286	302	282
126	326	298	318	314	314	360	372	348
151	396	366	380	380	380	432	448	412
175	468	432	444	448	440	494	520	484
200	540	498	514	518	506	566	590	554
226	610	574	582	588	576	636	674	628
251	686	638	654	660	656	710	750	706
275	750	704	730	728	718	778	838	778
300	822	768	804	798	784	842	910	846
325	904	854	890	881	880	928	1008	938
350	1002	944	988	978	978	1020	1118	1038
375	1112	1048	1104	1088	1094	1120	1246	1154
400	1302	1196	1454	1318	1268	1262	1440	1324
424	2338	1448	1656	1880	2344	1704	2252	2100

424 000 Ultimate Load

Machine head was not run down, and hence no observable bending occurred.

Readings continued on page following . . . .

COLUMN NO. 8 9 1 4

(Continued)

Observed Deformation over 100 Inches

	S. E. Flange				S. W. Flange			
Load	1	2	3	Av.	1	2	3	Av.
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
26	56	66	62	61	70	74	74	72
50	98	120	108	108	124	128	132	128
75	154	180	166	166	194	190	200	194
100	200	226	220	216	254	236	258	250
126	268	284	280	278	322	304	322	316
151	332	350	346	342	394	364	392	384
175	390	408	396	398	462	424	450	446
200	460	474	464	466	538	486	520	514
226	534	544	526	534	612	566	592	590
251	614	612	592	606	696	628	658	660
275	660	682	658	666	772	704	730	736
300	758	744	718	740	842	772	800	804
325	850	840	796	828	936	864	890	896
350	942	932	880	918	1036	980	980	998
375	1076	1040	982	1032	1152	1068	1098	1106
400	1260	1170	1122	1184	1320	1310	1266	1299
424	2278	1528	1652	1820	2132	1516	2156	1934

424 000 Ultimate Load

COLUMN NO. 8 9 1 5

Length: 15' - 4" Plain Steel

Observed Deformation

	150" Gauge Length over Entire Column				50" Gauge Length at Center of Col.				50" G.L. Bottom.	
Load	NW	NE	SE	SW	NW	NE	SE	SW	NW	SE
0	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
25	112	60	114	48	32	32		26	42	42
50	216	156	202	110	66	60	67	46	76	70
75	316	262	294	176	94	100	97	72	112	98
100	428	366	398	270	130	134	129	98	148	134
125	532	462	500	366	166	172	157	128	182	168
150	652	582	616	470	206	212	197	166	222	212
175	782	704	742	584	244	254	233	194	264	250
200	900	820	860	690	280	288	273	228	302	294
225	1032	946	990	806	328	332	313	266	346	340
250	1168	1084	1124	926	364	382	359	298	388	386
275	1330	1244	1270	1060	418	438	403	342	434	438
300	1498	1412	1422	1182	478	500	449	376	482	494
325	1702	1620	1576	1278	532	578	489	404	538	552
350	2020	2002.	1760	1338	640	752	515	396	602	658

368 000 Ultimate Load

One angle of SW flange was about 1/4 inch shorter than the other angles of the column, resulting in lower carrying capacity in that flange and consequently for entire column.

COLUMN NO. 8 9 1 6

Length: 15' – 4" Plain Steel

Observed Deformations

	150" Gauge Length over Entire Column				50" Gauge Length at Center of Column				50" G.L. Bottom	
Load	NW	NE	SE	SW	NW	NE	SE	SW	NW	SE
0	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
25	114	50	94	66	38	18	22	26	46	22
50	214	128	170	146	68	48	46	60	76	50
75	314	208	244	230	102	80	74	92	104	90
100	418	294	338	342	134	112	98	124	140	120
125	528	390	430	450	168	146	130	158	174	156
150	640	492	536	562	206	182	170	192	208	182
175	768	600	654	684	246	222	208	212	246	232
200	892	694	770	812	288	250	248	272	286	268
225	1030	802	892	944	334	294	294	338	326	306
250	1186	918	1024	1084	384	340	338	370	370	348
275	1336	1038	1154	1232	434	390	386	414	420	398
300	1506	1256	1304	1412	488	436	434	470	476	456
325	1692	1414	1458	1598	548	486	484	532	534	520
350	1954	1602	1616	1846	644	546	544	606	614	608
375	2634	1676	1794	2424	906	580	582	876	754	748

376 000 Ultimate Load

Column deflected to north of east, local failure (crimping) occuring finally in panel next below center of column.

COLUMN NO. 8 9 1 7

Length: 15' – 4" Core Section

Mixture: 1 : 2 : 4 Age: 61 Days

Observed Deformation over 150 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	56	68	74	74	74	74	74	58
50	114	128	130	126	136	130	134	108
75	172	212	198	206	208	192	194	162
100	254	296	274	290	298	270	254	234
125	326	372	354	374	376	336	328	294
150	412	468	482	486	482	422	412	362
175	490	558	556	582	584	518	504	460
200	584	662	642	688	690	610	598	542
225	662	748	740	800	802	722	704	626
250	754	848	820	910	908	826	796	716
275	844	950	920	998	1028	918	894	810
300	960	1062	1004	1156	1158	1048	1022	906
325	1060	1184	1120	1276	1288	1154	1134	994
350	1166	1294	1220	1414	1416	1276	1260	1104
375	1290	1432	1332	1560	1590	1408	1402	1224
400	1414	1572	1444	1732	1742	1552	1554	1346
425	1754	1752	1508	1958	1986	1772	1748	1466

468 500 Ultimate Load

Upper two feet of column very generally crushed. Especially marked at and above 2nd tie plate from top. Column bent to the north and West.

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 1 8

Length: 15' – 4" Core Section.

Mixture: 1 : 2 : 4 Age: 59 Days

Observed Deformations over 150 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	34	72	92	88	70	56	28	24
50	90	126	152	152	150	114	82	68
75	158	188	214	208	190	178	156	130
100	214	252	270	284	268	240	220	200
125	296	324	334	362	352	326	298	276
150	380	400	418	430	450	418	376	348
175	442	472	510	510	536	502	484	440
200	548	552	578	578	594	582	566	532
225	638	642	670	632	670	700	676	630
250	730	730	756	730		794	766	726
275	832	828	852	816	880	914	880	836
300	914	918	980	912	966	998	960	924
325	1018	1002	1068	974	1070	1120	1078	1034
350	1118	1106	1154	1036	1172	1212	1176	1146
375	1214	1196	1246	1154	1272	1334	1300	1256
400	1308	1314	1356	1250	1386	1446	1410	1360
425	1432	1416	1470	1404	1506	1578	1536	1488
450	1548	1544	1594	1510	1616	1708	1664	1612
475	1710	1662	1736	1614	1746	1870	1824	1986
500	1950	1930	1968	1876	1976	2084	2068	2054

532 200 Ultimate Load

Bottom of column crushed on all sides. Signs of crushing also at several levels near center of column, but top shows few evidences of distress.

NOTE: NW,NE,SE, and SW readings were taken on concrete faces.

COLUMN NO. 8 9 2 0

Length: 19' – 4" Plain steel.

Observed Deformation

	200" Gauge Length over Entire Column				60" Gauge Length at Center of Col.				60" G.L. Bottom	
Load	NW	NE	SE	SW	NW	NE	SE	SW	NE	SW
0	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
25	118	73	154	102	30	20	34	68	16	46
50	232	158	182	340	68	42	70	112	38	88
75	366	370	414	470	98	70	86	150	62	122
100	492	400	548	510	134	102	142	192	102	182
125	624	540	712	738	168	138	186	230	138	200
150	772	686	854	890	210	174	232	270	174	240
175	918	842	1036	1042	252	222	284	322	240	284
200	1080	1024	1214	1226	294	264	338	372	268	332
225	1266	1210	1414	1404	346	314	396	430	324	384
250	1438	1398	1598	1590	398	348	454	486	382	434
275	1630	1598	1812	1790	454	422	516	550	444	494
300	1830	1810	2032	2002	508	482	584	610	482	556
325	2042	2026	2252	2222	564	544	658	682	536	614
350	2256	2276	2552	2464	618	614	762	798	594	692

374 200 Ultimate Load

Column bent to north and west, local failure occurring in the two center panels.

COLUMN NO. 8 9 2 1

Length: 19' – 4" Plain Steel

Observed Deformation

	200" Gauge Length over Entire Column				60" Gauge Length at Center of Col.			60" G.L. Bottom	
Load	NW	NE	SE	SW	NW	SE	SW	NE	SW
0	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
25	104	140	92	108	0	36	50	41	40
50	228	236	202	224	18	66	86	80	80
75	358	348	306	360	96	98	132	124	118
100	496	468	426	498	100	136	168	164	146
125	642	598	554	642	116	162	220	214	190
150	804	742	700	804	158	210	276	266	234
175	968	888	852	984	220	260	326	326	284
200	1156	1048	1008	1174	304	304	386	382	332
225	1372	1202	1174	1380	368	354	458	446	390
250	1582	1364	1340	1612	418	404	532	510	446
275	1828	1522	1532	1856	506	462	616	580	512
300	2068	1710	1708	2126	590	526	714	648	574
325	2348	1930	1900	2480	668	596	848	724	640

345 200 Ultimate Load

Failed by bending slightly to north of east.

COLUMN NO. 8 9 2 2

Length: 19' – 4" Core Section.

Mixture: 1 : 2 : 4 Age: 60 Days

Observed Deformations over 200 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	62	82	88	92	80	66	50	38
50	140	160	174	180	168	148	114	126
75	226	254	260	278	242	230	202	210
100	340	348	366	392	362	340	312	336
125	434	444	456	498	466	428	390	434
150	540	478	572	624	588	548	504	554
175	656	676	692	734	700	646	614	670
200	774	814	838	884	830	786	730	880
225	902	934	932	1000	944	910	850	902
250	1060	1090	1096	1148	1102	1060	1034	1068
275	1236	1242	1254	1300	1248	1180	1148	1226
300	1388	1400	1410	1452	1398	1364	1312	1376
325	1560	1572	1576	1624	1538	1508	1450	1562
350	1754	1760	1748	1994	1720	1648	1612	1742
375	1938	1934	1942	2000	1900	1856	1740	1904
400	2126	2120	2096	2176	2060	1896	1892	2082

491 400 Ultimate Load

Bending failure. Crushing on West face top and bottom and on East face at about center of length.(between 6th and 7th tie plate from top)

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 2 3

Length: 19' – 4" Core Section.

Mixture: 1 : 2 : 4 Age: 60 Days

Observed Deformations over 200 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	76	96	118	84	72	56	52	50
50	154	184	222	152	132	110	140	132
75	234	296	284	260	230	210	212	226
100	354	376	366	356	312	300	314	306
125	450	482	482	466	410	394	404	402
150	538	590	584	574	526	504	514	492
175	648	690	696	688	646	628	630	612
200	764	730	816	812	774	786	754	748
225	864	950	930	942	900	902	884	848
250	974	1046	1048	1054	1022	1026	992	968
275	1112	1178	1180	1184	1160		1136	1100
300	1236	1284	1308	1310	1248		1256	1228
325	1356	1432	1512	1426	1422	1426	1396	1344
350	1492	1576	1564	1586	1572	1550	1534	1474
375	1616	1706	1704	1718	1712	1688	1676	1604
400	1752	1840	1836	1870	1862	1828	1840	1748
425	1920	2012	1994	2042	2028	1962	2000	1906
450	2112	2218	2204	2250	2206	2062	2176	2086
475	2410	2530	2470	2536	2364	2162	2270	2348

495 500 Ultimate Load

Very general failure. Crushing most noticeable at top and bottom.

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 2 5

Length: 10' – 0"

Core Section.

Mixture: 1 : 1 : 2

Age: 61 Days

Observed Deformations over 100 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	28	38	46	50	42	40	34	36
50	68	80	90	92	80	74	60	58
75	100	114	122	128	114	114	100	98
100	144	162	172	172	146	154	136	138
125	178	188	198	208	184	194	176	172
150	214	230	244	244	226	234	220	200
175	266	278	294	290	276	286	266	242
200	308	322	334	330	318	338	314	286
225	358	376	386	380	368	392	366	336
250	408	416	430	424	416	434	418	384
275	462	470	482	478	466	494	466	434
300	508	516	536	530	522	548	522	490
325	560	566	612	592	572	602	572	538
350	612	622	640	648	630	658	634	598
375	654	672	694	696	678	714	684	640
400	716	732	750	756	742	784	752	702
425	770	782	810	818	790	836	800	760
450	836	840	866	884	846	896	866	824
475	888	900	930	948	906	970	930	880
500	956	960	982	1014	968	1010	994	946
525	1018	1034	1060	1094	1042	1100	1060	1026
550	1088	1110	1132	1176	1122	1178	1144	1104
575	1182	1210	1238	1286	1232	1290	1240	1212
600	1298	1336	1370	1374	1476	1382	1342	1368
625	1564	1602	1596	1672	1586	1680	1684	1600

636 000 Ultimate Load

Failed by general crushing of upper fifteen inches of column, most marked ten inches from top.

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 2 6

Length: 10' - 0" Core Section.

Mixture: 1 : 1 : 2 Age: 60 Days

Observed Deformations over 100 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	28	52	54	48	28	24	18	28
50	70	112	104	98	72	54	52	56
75	122	168	154	152	118	98	90	100
100	162	210	202	204	164	124	118	132
125	210	250	236	240	202	168	168	176
150	256	306	294	290	250	214	202	228
175	308	352	334	338	296	260	254	278
200	350	406	394	382	348	310	304	318
225	408	450	432	422	384	366	358	372
250	450	500	478	478	432	410	392	422
275	502	564	530	538	490	464	454	478
300	564	612	580	592	546	532	530	534
325	618	670	640	660	602	592	574	598
350	666	716	688	714	658	648	638	652
375	716	772	750	772	719	706	694	706
400	774	838	798	832	774	766	746	776
425	828	898	862	896	838	826	798	832
450	878	936	910	942	894	892	868	894
475	936	996	978	1010	954	948	930	956
500	998	1066	1030	1072	1016	1024	982	1034
525	1066	1132	1102	1158	1096	1096	1066	1110
550	1134	1222	1158	1218	1156	1170	1144	1190
575	1236	1326	1272	1348	1284	1298	1258	1308
600	1362	1446	1412	1488	1416	1428	1368	1438
625	1616	1680	1612	1712	1642	1652		1666
650	2060	2108	2048	2126	1846	2058		2106

655 000 Estimated Ultimate Load

Failure did not occur, but was very imminent.

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 2 7

Length: 10' – 0" | Core Section

Mixture: 1 : 3 : 6 | Age: 59 Days

Observed Deformations over 100 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	26	50	66	66	22	14	18	24
50	90	112	116	110	78	54	40	52
75	134	180	174	186	122	112	98	98
100	184	228	224	254	174	152	116	144
125	226	300	330	310	224	204	160	190
150	302	376	358	376	296	254	210	242
175	354	450	422	448	362	298	260	302
200	426	516	490	520	414	366	310	364
225	490	590	566	600	486	418	366	422
250	556	670	632	660	548	492	422	480
275	624	742	704	748	614	554	480	552
300	700	824	778	834	692	630	560	632
325	766	900	844	900	758	685	610	700
350	854	990	926	986	830	768	670	764
375	928	1080	998	1060	912	832	738	846
400	1016	1174	1096	1166	1008	914	820	928
425	1096	1260	1182	1254	1082	1014	906	1026
450	1220	1364	1306	1366	1188	1108	1004	1118
475	1386	1506	1486	1504	1320	1230	1128	1252
500	1530	1786	1810	1756	1594	1462	1380	1512

516 000 Ultimate Load

Failed by local crushing at several levels near center of column, caused by bending.

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 2 8

Length: 10' – 0" Core Section.

Mixture: 1 : 3 : 6 Age: 60 Days

Observed Deformations over 100 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	44	58	62	56	48	36	30	38
50	82	108	108	104	92	74	60	70
75	139	162	162	166	146	132	110	126
100	202	224	202	228	206	188	160	186
125	252	274	246	280	262	242	220	228
150	312	322	292	340	322	306	276	292
175	374	386	340	400	384	372	338	356
200	438	458	388	472	454	440	396	428
225	496	514	444	540	520	506	460	488
250	556	568	498	620	590	576	526	548
275	618	642	562	692	656	644	594	626
300	678	700	620	770	738	720	638	686
325	746	772	684	850	812	792	724	758
350	810	840	760	948	910	882	798	826
375	882	916	828	1028	986	956	882	906
400	942	994	916	1134	1074	1052	954	972
425	1018	1088	1004	1222	1166	1122	1042	1050
450	1096	1172	1110	1362	1248	1258	1134	1168
475	1166	1292	1254	1568	1310	1396	1240	1222
500	1306	1566	1548	1720	1660	1652	1538	1406
530			4072				3000	

530 500 Ultimate Load

Failed by crushing at lower end. Sharp bend produced about eighteen inches from base on running down machine head.

NOTE: N, E, S, and W readings were taken on concrete faces.

COLUMN NO. 8 9 2 9

Length: 10' – 0" 2" Fireproofing

Mixture: 1 : 2 : 4 Age: 60 Days.

Deformations Observed over 100 Inches

Load	NW	N	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	48	60	38	10	24	28	40
50	84	98	58	18	40	56	72
75	128	138	94	66	80	98	114
100	160	168	132	90	120	132	154
125	202	212	156	128	150	174	200
150	242	252	190	168	186	220	240
175	292	292	218	192	224	266	286
200	334	326	256	236	240	308	336
225	382	378	288	292	318	364	392
250	426	420	338	332	364	406	436
275	480	464	370	372	418	466	496
300	522	502	416	420	466	516	546
325	594	542	452	466	512	572	606
350	628	586	498	530	566	628	656
375	680	644	540	572	618	686	706
400	726	682	600	624	668	732	756
425	782	740	638	672	720	800	816
450	834	780	688	732	774	856	872
475	900	850	742	786	832	920	936
500	956	900	804	846	898	982	994
525	1024	978	856	896	958	1056	1074
550	1086	1032	916	956	1020	1126	1140
575	1190	1134	988	1024	1090	1224	1226
600	1300	1240	1086	1128	1208	1340	1356
0	254	238	126	182	226	300	322

No failure under repetition of load four times.

Only two small cracks developed during test.

COLUMN NO. 8 9 3 0

Length: 10' - 0" 2" Fireproofing

Mixture: 1 : 2 : 4 Age: 60 Days

Observed Deformations over 100 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	36	64	80	78	54	28	8	12
50	70	96	140	126	100	64	38	34
75	118	166	200	178	142	88	62	62
100	148	206	242	216	202	142	100	96
125	188	250	296	282	250	176	138	124
150	222	290	344	340	304	226	172	160
175	274	344	400	410	374	276	232	198
200	314	394	456	468	432	336	276	244
225	360	440	500	516	478	384	324	280
250	410	484	568	588	556	450	378	334
275	460	540	610	630	604	506	430	380
300	512	580	666	688	666	564	484	428
325	566	634	720	746	726	616	540	484
350	618	662	776	810	786	676	592	532
375	678	740	836	884	858	748	640	586
400	726	792	890	946	918	812	706	632
425	796	862	948	1014	990	866	768	696
450	854	910	1022	1094	1072	942	824	754
475	918	978	1092	1156	1144	1006	894	818
500	978	1042	1174	1248	1240	1104	968	878
525	1058	1132	1268	1340	1336	1182	1058	962
550	1132	1198	1354	1442	1436	1290	1134	1036
575	1242	1314	1458	1556	1570	1394	1260	1148
600	1362	1422	1608	1732	1768	1582	1404	1270
625	1598	1608	1824	2010	2120	1916	1714	1534

630 700 Ultimate Load

Load dropped off very rapidly till 565 000 pounds was reached which was held for 12 - 15 minutes and was probably the core strength. Outer shell gradually broke up and finally split off for the lower three feet of the column.

COLUMN NO. 8 9 3 1

Length: 10' – 0" 2" Fireproofing

Mixture: 1 : 2 : 4 Age: 60 Days

Observed Deformations over 100 Inches

Load	NW	N	NE	E	SE	S	SW	W
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
25	–	44	48	44	40	16	12	20
50	–	82	80	70	72	42	36	38
75	–	118	114	96	106	74	70	100
100	–	144	140	126	128	106	108	142
125	180	162	180	164	176	148	148	180
150	212	226	220	214	218	188	192	230
175	256	266	256	254	252	230	242	284
200	296	308	298	280	304	282	286	324
225	356	358	340	324	344	332	344	376
250	400	400	382	376	396	376	386	412
275	460	456	426	416	444	430	444	462
300	512	494	478	472	502	480	502	498
325	578	554	520	512	562	538	566	544
350	638	614	582	574	618	596	636	604
375	718	682	640	620	674	656	712	684
400	790	736	706	694	744	718	778	
425	874	810	762	744	814	782	844	
450	944	874	834	816	884	850	908	
475	1036	956	898	874	948	920	996	
500	1116	1026	978	956	1028	996	1078	
525	1232	1118	1056	1022	1110	1068	1178	
550	1348	1214	1158	1118	1214	1178	1284	
575	1510	1360	1266	1232	1328	1278	1436	
600	1690	1500	1428	1368	1478	1450	1618	
625	2046	1842	1690	1614	1714	1856	1942	

635 700 Ultimate Load

Load dropped quite rapidly to 570 400 pounds and then rose very gradually to 572 800 pounds, which may be considered the strength of the column core. Outer shell finally peeled off at the middle of the column length.

COLUMN NO. 8 9 3 3

Length: 10' – 0" 3/4 % Spiral.

Mixture: 1 : 2 : 4 Age: 60 Days.

Observed Deformation over 95.6 Inches

Load	N	E	S	W
0	0.0000	0.0000	0.0000	0.0000
25	70	52	28	28
50	90	96	46	58
75	132	136	98	94
100	176	184	114	122
125	204	228	148	158
150	246	282	196	202
175	298	328	234	244
200	342	372	280	290
225	386	434	330	332
250	432	490	378	376
275	492	544	430	434
300	536	600	486	476
325	596	652	528	526
350	656	720	594	584
375	690	774	632	628
400	748	834	688	682
425	806	894	740	736
450	864	956	802	786
475	934	1034	864	840
500	1004	1108	938	906
525	1084	1202	1008	976
550	1164	1254	1080	1046
575	1308	1428	1168	1138
600	1528	1648	1338	1276
0	458	390	330	220

Maximum load applied five times disclosed stable stress conditions in column.

COLUMN NO. 8 9 3 4

Length: 10' – 0"   3/4 % Spiral

Mixture: 1 : 2 : 4   Age: 60 Days.

Observed Deformation over 100 Inches

Load	NW	NE	SE	SW
0	0.0000	0.0000	0.0000	0.0000
25	40	68	22	24
50	66	112	54	46
75	98	156	82	76
100	134	208	126	106
125	180	254	170	140
150	206	302	222	170
175	260	354	272	224
200	300	410	334	266
225	346	460	386	320
250	404	520	446	362
275	460	588	510	422
300	490	650	570	470
325	558	718	648	544
350	606	790	708	592
375	670	858	802	666
400	736	934	878	736
425	794	1012	958	796
450	848	1096	1046	866
475	930	1184	1146	962
500	1006	1310	1266	1054
525	1106	1430	1380	1156
550	1216	1590	1548	1278
575	1356	1766	1686	1476
600	1544	2060	2020	1648
0	510	928	932	476

Stable stress conditions in column

COLUMN NO. 8 9 3 5

Length: 10' – 0" 3/4 % Spiral

Mixture: 1 : 2 : 4 Age: 59 Days

Observed Deformations over 100 Inches

Load	N	E	S	W
0	0.0000	0.0000	0.0000	0.0000
25	42	42	30	34
50	70	80	58	66
75	112	126	102	100
100	142	162	134	134
125	188	200	176	176
150	222	242	212	212
175	280	292	262	256
200	320	348	318	306
225	372	398	360	350
250	410	452	414	394
275	462	502	466	450
300	512	556	518	492
325	568	610	574	544
350	612	666	634	594
375	664	722	686	636
400	718	790	738	690
425	772	844	800	736
450	824	904	858	788
475	864	966	914	836
500	928	1022	972	896
525	990	1114	1054	956
550	1056	1176	1120	1024
575	1132	1282	1222	1086
600	1236	1416	1336	1174
0	160	296	296	158

Maximum load applied three times.

COLUMN NO. 8 9 3 6

Length: 10' — 0" 3/4 % Spiral

Mixture: 1 : 2 : 4 Age: 60 Days.

Observed Deformations over 95.8 Inches

Load	N	E	S	W
0	0.0000	0.0000	0.0000	0.0000
25	54	14	20	40
50	90	58	42	74
75	128	94	90	110
100	170	140	116	142
125	216	180	168	188
150	260	222	204	226
175	314	262	260	278
200	352	318	292	318
225	412	362	346	376
250	450	416	402	414
275	496	460	444	464
300	546	522	506	518
325	608	580	570	574
350	656	640	626	628
375	722	694	690	676
400	776	754	746	734
425	832	806	802	784
450	894	874	862	848
475	968	952	940	908
500	1036	1020	1008	978
525	1102	1116	1098	1040
550	1184	1216	1184	1134
575	1308	1382	1342	1240
600	1548	1600	1522	1432
625	1872	1978	1822	1674
0	768	782	662	600

Stable stress condition in column

COLUMN NO. 8 9 3 7

Length: 10' – 0" 1 % Spiral

Mixture: 1 : 2 : 4 Age: 60 Days

Observed Deformations over 100 Inches

Load	N	E	S	W
0	0.0000	0.0000	0.0000	0.0000
25	40	40	34	36
50	72	79	68	72
75	110	96	108	100
100	152	132	146	142
125	194	176	182	180
150	234	220	216	216
175	286	268	264	260
200	334	324	308	304
225	382	370	358	356
250	426	428	404	398
275	474	484	458	450
300	532	538	510	492
325	584	596	564	540
350	632	650	618	592
375	688	712	672	646
400	734	766	724	696
425	796	834	788	748
450	832	898	848	802
475	918	966	908	864
500	986	1048	978	926
525	1050	1122	1042	964
550	1136	1228	1130	1028
575	1240	1366	1230	1090
600	1424	1584	1390	1210
0	306	470	294	156

Maximum load applied three times.

COLUMN NO. 8 9 3 8

Length: 10' – 0" 1 % Spiral

Mixture: 1 : 2 : 4 Age: 59 Days.

Observed Deformations over 99.4 Inches

Load	NW	NE	SE	SW
0	0.0000	0.0000	0.0000	0.0000
25	22	12	52	64
50	46	38	84	98
75	88	72	130	142
100	154	104	166	192
125	172	144	216	242
150	212	194	270	294
175	252	234	316	340
200	294	278	372	398
225	338	332	430	456
250	374	382	460	514
275	408	424	544	576
300	438	472	614	634
325	474	534	690	692
350	488	592	756	752
375	498	648	824	818
400	536	708	890	890
425	578	768	944	964
450	628	836	1024	1050
475	672	904	1086	1118
500	730	996	1166	1196
525	792	1096	1260	1284
550	836	1196	1348	1392
575	912	1346	1480	1518
600	1076	1580	1652	1740
0	74	464	502	558

Stable stress condition in column

COLUMN NO. 8 9 3 4

Second Test (Made at Lehigh University)

Length: 10' – 0" 3/4 % Spiral

Mixture: 1 : 2 : 4 Age: 138 Days.

	Observed Deformations over 100 Inches				Av. Unit
Load	N	E	S	W	Deformation
3	0.0000	0.0000	0.0000	0.0000	0.000000
55	60	92	88	72	78
107	154	212	184	140	172
205	336	412	398	332	370
303	510	606	544	514	544
403	618	794	796	684	724
508	882	998	984	876	936
528	928	1044	1036	1044	1012
560	974	1102	1086	1092	1064
607	1054	1190	1174	1166	1146
632	1104	1244	1226	1206	1196
660	1162	1316	1290	1262	1258
678	1222	1410	1374	1310	1328
702	1394	1564	1502	1464	1480
726	1616	1766	1676	1660	1680
752	1846	2002	1892	1914	1914
773	2162	2306	2154	2160	2196
800	2786	2922	2724	2680	2778
825	3680	3752	3224	3388	3510
850	4820	4728	4004	4290	4460
856	5202	4962	4128	4584	4720
Load of 840 000 applied and released three times					
840	6380	5886	4688	4904	5464
2	5010	4476	3378	3600	4116

Column was very close to ultimate load. Marked spalling and cracking occurred at top of column and also (less marked) at bottom of column.

COLUMN NO. 8 9 3 5

Second Test (Made at Lehigh University)

Length: 10' — 0" 3/4 % Spiral

Mixture: 1 : 2 : 4 Age: 152 Days

	Observed Deformation over 100 Inches				Av. Unit
Load	N	E	S	W	Deformation
3	0.0000	0.0000	0.0000	0.0000	0.000000
103	204	196	164	182	186
203	440	410	378	386	404
303	638	610	570	578	600
402	832	798	744	752	782
506	1024	988	918	930	964
529	1072	1026	956	972	1006
555	1122	1066	996	1016	1050
580	1170	1106	1040	1056	1092
602	1202	1144	1074	1094	1128
630	1254	1198	1126	1138	1180
655	1310	1274	1198	1190	1242
679	1392	1388	1298	1250	1332
702	1540	1566	1450	1356	1478
728	1730	1758	1630	1490	1652
750	1922	2040	1820	1626	1852
772	2226	2402	2100	1800	2132
799	2780	3148	2564	2098	2648
825	3774	4078	3308	2628	3448
Load of 830 000 applied and released five times					
828	4876	5870	4242	3228	4554
2	3568	4542	2934	1924	3242

Concrete spalled badly near bottom of column but the ultimate load was not reached.

COLUMN NO. 8 9 3 6

Second Test

Length: 10' – 0"   3/4 % Spiral

Mixture: 1 : 2 : 4   Age: 163 Days

	Observed Deformation over 95.8 Inches				Av. Unit
Load	N	E	S	W	Deformation
0	0.0000	0.0000	0.0000	0.0000	0.000000
60	56	106	148	78	101
100	122	192	230	140	180
150	216	296	340	228	282
200	312	412	460	334	398
250	418	538	580	434	515
300	514	652	702	530	628
350	618	774	810	626	741
400	722	892	930	730	856
450	828	1012	1048	840	976
500	936	1132	1160	932	1090
525	992	1198	1222	988	1160
550	1046	1306	1290	1022	1220
575	1126	1442	1400	1062	1317
600	1234	1624	1536	1116	1440
0	124	490	304	– 40	230

COLUMN NO. 8 9 3 7

Second Test (Made at Lehigh University)

Length: 10' – 0" 1 % Spiral

Mixture: 1 : 2 : 4 Age: 159 Days

Load	N	E	S	W	Av. Unit Deformation
	Observed Deformation over 100 Inches				
2	0.0000	0.0000	0.0000	0.0000	0.000000
55	106	78	86	110	96
103	196	168	170	210	186
200	408	362	372	420	390
304	618	570	586	632	602
400	798	750	762	808	780
499	980	924	934		952
525	1026	970	982		992
554	1078	1020	1034		1044
600	1152	1102	1108		1120
626	1198	1146	1158	1176	1170
652	1290	1212	1248	1264	11254
673	1392	1316	1366	1378	1364
702	1528	1462	1506	1510	1496
723	1694	1630	1668	1658	1662
752	1892	1840	1880	1842	1864
775	2182	2140	2176	2118	2154
804	2552	2538	2566	2460	2528
819	2848	2866	2870	2720	2836
829	3056	3108	3080	2900	3036
	Load of 830 000 applied and released three times				
830	3616	3068	3676	3310	3418
4	2262	1762	2478	2016	2130

No decided signs of failure were observed.

COLUMN NO. 8 9 3 7

Third Test (Made at Lehigh University)

Length: 10' – 0" Core Section (Spiral removed)

Mixture: 1 : 2 : 4 Age: 180 Days

Observed deformations over 100 Inches

Load	NW	NE	SE	SW	Av. Unit Deformation
0	0.0000	0.0000	0.0000	0.0000	0.000000
103	218	192	122	150	170
200	430	392	264	316	350
301	634	590	414	488	532
400	826	770	566	656	704
498	992	956	720	810	870
598	1206	1146	882	978	1052
698	1628	1496	1070	1192	1346
714	2332	2090	1282	1520	1806

714 000 Ultimate Load

Failed by buckling of steel and crushing of concrete

COLUMN NO. 8 9 3 8

Second Test (Made at Lehigh University)

Length: 10' – 0" 1 % Spiral

Mixture: 1 : 2 : 4 Age: 143 Days

Load	NW	NE	SE	SW	Av. Unit Deformation
	Observed Deformation over 99.6 Inches				
3	0.0000	0.0000	0.0000	0.0000	0.000000
58	96	112	88	82	94
106	198	208		176	194
206	396	404		362	388
303	582	584		536	568
410	782	794		724	766
510	946	986	940	890	940
532	996	1030	974	932	984
560	1038	1084	1014	972	1028
583	1080	1120	1058	1014	1068
608	1124	1166	1096	1056	1120
636	1176	1216	1140	1104	1160
666	1240	1306	1196	1152	1224
697	1362	1456	1296	1334	1336
728	1604	1682	1470	1422	1544
749	1824	1920	1658	1594	1748
776	2174	2264	1936	1874	2062
800	2688	2766	2300	2244	2500
825	3488	2844	2808	2818	3090
Load of 826 000 applied and released three times					
827	4484	4156	3300	3418	3840
2	3168	2830	2054	2154	2552

No decided signs of failure were observed.